Appendix A – Design Basis Manual

SR 520 PONTOON CONSTRUCTION FACILITY DESIGN-BUILD PROJECT DESIGN BASIS MANUAL

KIEWIT-GENERAL JV
HNTB Corporation

CONTENTS

REVISION LOG

Revision Number	Issue Date	Revision Details

PART 1.0 BASIS OF DESIGN – GENERAL

1.1 Objective

The objective of this manual is to capture the discipline specific design criteria and document the design decisions that were made for the basis of the proposal. Also, this document will include as-bid quantities for major permanent materials which will allow all designers to see what their design goals are. These quantities will be tracked throughout the design to ensure that the designers meet or beat the estimate. This document will also capture design innovations that were talked about during the proposal but not included in the proposal so that the designers are not revisiting ideas that have already been eliminated. This manual will be distributed to all team members of each discipline.

1.2 Design Overview

The design of the casting basin for the SR520 Pontoons will start upon receiving NTP-1 (estimated 1/15/10). The preliminary design effort is expected to run from mid-January 2010 to September 2010 to support the FEIS, JARPA, and permitting process. Upon receiving NTP-2 (expected when the ROD is issued on 10/31/2010), the final design will be completed. The Released for Construction (RFC) documents will be issued by December 31, 2010 to facilitate construction starting January 2011.

The approach to completing the casting basin design is to start the geotechnical investigations (consisting of a test pile program, additional borings, slope evaluation, and dewatering evaluation) upon receiving NTP-1. While the geotechnical investigations on the casting basin site are being completed, preliminary design will start on the casting basin gate, hydraulic control structure, and launch channel dolphins, along with the offsite roadway work. Once the geotechnical investigations have been completed and evaluated, the remaining casting basin work can proceed (anticipated spring 2010). Applications for the local permits will be assembled once the preliminary casting basin design work has been partially advanced (expected summer 2010).

The pontoons are being designed and sealed by WSDOT. Upon receiving NTP-2, the engineering for the temporary mooring and marine outfitting of the pontoons will proceed, along with the design of the precast options for the interior pontoon walls and anchor galleries.

1.3 Project Description

1.3.1 Basin

1.3.1.1. Design and construct a pontoon casting facility at the Grays Harbor, Washington site.

1.3.2 Pontoons

1.3.2.1. Construct longitudinal, cross and supplemental stability pontoons that can accommodate the infrastructure for a 4-lane replacement of the SR 520 floating bridge.

1.3.2.2. Construct pontoons that are compatible with the planned 6-lane expansion and potential future 6-lane plus high-capacity transit (HCT) floating bridge.

1.3.2.3. Develop temporary moorage to store completed pontoons until acceptance by WSDOT.

1.3.2.4. Tow completed pontoons to WSDOT provided facility.

1.3.3 Site

1.3.3.1. Develop infrastructure of Grays Harbor site to support the pontoon casting facility.

1.3.3.2. Design and construct local street improvements and a railroad crossing.

1.4 Major Cost Drivers / Key Risk Areas

1.5 The ATC Process

The Owner's procurement package included Basic Configuration drawings that could be changed by means of a formal Alternate Technical Concept (ATC) approval process. ATC's that require a deviation cannot be included in the Proposal unless they have been approved by WSDOT. A deviation is defined as a documented decision granting approval at project-specific locations to differ from the design level specified in the Design Manual, Chapter 1100. Proposed ATC's must not have an adverse effect on the project Quality and this is determined solely by WSDOT. Proposed ATC's most likely to receive favorable consideration are those that are consistent with WSDOT's values and Project goals, and those that maximize efficiency, incorporate technical innovation, increase expected life cycles, minimize environmental impacts, or otherwise improve the quality of the Project. A number of ATC's were proposed, accepted by WSDOT and included in the Basis of Design for estimating purposes. The approved ATC's that are included in the Proposal are summarized as follows:

- ***ATC 1 – Optimize Casting Basin Size***

  This ATC adjusted the casting basin size from a capacity to cast eight pontoons per cycle to a more optimal size of four to six pontoons per cycle. Kiewit-General's history of constructing this type of pontoon validates productions that allow for faster cycle times and thus, optimizing the casting basin size.

- ***ATC 4 – Remove Battered Walls***

  This ATC altered the exterior walls by removing the batter. The design would provide a 1'-2" wide wall from the top to the bottom of the wall in lieu of a battered wall that went from 1'-0" wide at the top of the wall to 1-4 1/2" wide at the bottom of the wall.

- ***ATC 8 – Rebar Connection at the Exterior Wall to Top Deck***

  This ATC altered the typical exterior wall to deck slab rebar connection to improve quality and efficiency of the rebar installation.

- ***ATC 9.2 – Precast Anchor Galleries***

  This ATC used precast components in the lower section of the anchor galleries for both the longitudinal and cross pontoons. The components that are precast include the interior lower lift walls and the mid-level slabs. To support the mid-level slabs, corbels will be added to both the cast-in-place walls and the precast panels. The upper walls of thechor galleries will be cast-in-place to insure a water tight joint is constructed.

- ***ATC 10 – Hydroseeded Berm***

  This ATC stockpiled excavated spoils from the casting basin on the west side of the property. The slopes of the berm will be no steeper that 3:1 and will be covered with a standard WSDOT hydroseed mix for erosion control. The overall height of the berm may be up to thirty feet tall. This ATC reduces the impacts to the community by eliminating up to 13,000 trucks from entering and leaving the project as well as reducing the cost to construct the facility.

- ***ATC 11 - Delivery Cycle***

  This ATC alters the delivery of the pontoons to optimize the efficiency of the casting basin and delivers the pontoons with more schedule regularity

1.6 Innovations, Risk Mitigation, and Three-Dimensional Modeling

1.7 Deliverables

Discipline	RFP Reference	Deliverable
Management	2.1.3.4.3.1	Document Control Work Plan
	2.1.2.2.5	Pre-contract Meeting
	2.5.4.2	Kick-Off Meeting
Geotechnical	2.6.2.2	Monitoring Wells
	2.6.3.2	Subsurface Investigation Plan
	2.6.3.4	Geotechnical Instrumentation Plan
	2.6.6.1	Calculations Verification Submittal
	2.6.3.8	Geotechnical Report
		Geotechnical Specifications
		Operations and Maintenance Manual
	2.7.6.1 & 2.7.6.2	Pavement Memo
Civil	2.5.6.2	MOT Plan for Surveying
	2.11.6.1	Roadway Design Technical Memo
	2.7.6.1	HMA Mix Design
	2.7.6.2	City Street/Sidewalk Pavement
Environmental	2.8.1.1	Support of Final EIS
	2.8.3.2.1	Draft Environmental Compliance Plan
	2.8.3.2.1	Final Environmental Compliance Plan
	2.8.3.2.3.1. Para 3	Final TESC Plan
	2.8.3.2.3.5	Monitoring Plan for NPDES S & G General Permit
	2.8.4	Commitments Database
	2.8.4.2.1	404 Permit Support
	2.8.4.2.1	401 Permit Support
	2.8.4.2.1	Hydraulic Project Approval (WDFW) Support
	2.8.4.2.1	Coastal Zone Mgmt Act Consistency Determ Support
	2.8.4.2.1	Aquatic Lands Use Authorization Support
	2.8.4.2.1	Shoreline Substantial Development Permits Support
	2.8.4.2.1	Critical Area Compliance Support
	2.8.4.2.1	Private Aids to Navigation (PATON) Permit (USCG)
	2.8.4.2.1	Dredge Disposal Site Use Authorization (DNR)

Discipline	RFP Reference	Deliverable
	2.8.4.2.1	Air Quality Notice of Construction
	2.8.4.2.1	Street Use Permit – City of Aberdeen
	2.8.4.2.1	Street Use Permit – City of Hoquiam
	2.8.4.2.1	Building Permit – City of Aberdeen
	2.8.4.2.1	NDPES Municipal Permit
	2.8.5.1	JARPA Support
	2.8.4.3.10	Dredged Material Management and Disposal Plan
	2.8.5.2	Restoration Plans for Temp Shoreline Impacts
	2.8.4.2.1	NPDES Sand and Gravel General Permit
	2.8.4.3.6	Biological Assessment Support
	2.8.3.2.4.3.1	Environmental Commitment close-out Report
Utilities	2.10.8.12.2	Utility As-Built Plans
Document Control	2.12.3.1	Design Documentation Pkg and Proj File
	2.12.3.2.2	Local Streets Channelization Plan
	2.12.5.4	Comprehensive Operations & Maintenance Manual
	2.12.6.1	Final Project Design Documentation
Casting Facility	2.13.3.2.1.6.7	Utility Easement on Casting Facility Site
	2.13.6.1.1	Casting Facility Preliminary Design Submittal
	2.13.6.1.1	Coastal Engineering Report (Part of Prelim Submittal)
	2.13.6.1.2	Casting Facility Final Design ?Submittal
	2.13.6.1.3	RFC Document Submittal
	2.13.6.4	Casting Facility Operations & Maintenance Manual
Pontoons	2.14.1.1.1	Design Pontoon Launching and Towing
	2.14.1.1.1	Geometry Control Plan
	2.14.1.1.1	Design Hatches & Doors
	2.14.1.1.1	Design Access/Maintenance Walkway
	2.14.1.1.1	Design of Mooring /Towing Bollards & Fendering Sys
	2.14.1.1.1	Completions of Integrated Pontoon Drawings
	2.14.1.1.1	Design of Post-Tensioning System
	2.14.1.1.1	Design Precast Anchor Galleries

Discipline	RFP Reference	Deliverable
	2.14.6.1.1	Preliminary Design Plans
	2.14.6.1.2	Final Design Plans
Mooring	2.15.6.3	Pontoon Monitoring System O & M Manual
Drainage	2.17.6.2	Drainage Existing conditions Survey
	2.17.6.3	Design Calculations
	2.17.6.4	Draft Hydraulic Report
	2.17.6.7	Final Hydraulic Report
	2.17.6.8	Drainage Operations & Maintenance Manual
	2.17.6.9	Monitoring Plan
Electrical	2.19.5.1	Preliminary Design Submittal
	2.19.5.1.1	Power Study
	2.19.5.2	Final Design Submittal
	2.19.5.3	RFC Electrical Plans
Signals	2.20	Temporary Signals
	2.20	Haul Route Signals
	2.20.6.2	Preliminary Signal Plans
	2.20.6.3	Final Signal Plans
	2.20.6.4	RFC Signal Plans
Signing	2.22.4.1	Preliminary Design Submittal
	2.22.4.2	Permanent Signing Plans (Final Design Submittal)
	2.22.5.1	Existing Sign Inventory
Pvmt Markings	2.23.5.1	Pavement Marking Plans (Preliminary Submittal)
	2.23.5.2	Pavement Parking Plans (Final Submittal)
MOT	2.24.3	Traffic Analysis Plan
	2.25.3.1	Traffic Management Plan
	2.25.3.2	Traffic Incident Management Plan
	2.25.4.1.2	MOT Plans
	2.25.5.4	Temporary Signal Plans

Discipline	RFP Reference	Deliverable
	2.25.5.5	Temporary Illumination Plans
Railroad	2.26.6.1	Railroad Design Drawings
Quality	2.31.1	Draft QMP
	2.31.1	Final QMP
	2.31.6.2	Executive Management Reviews and Audits
	2.31.6.3	Review Documents
	2.31.6.4	QA/QC Control Documentation

1.8 Technical Tasks

Part 1 – N/A
Part 2 – Site Roadway, Civil, Utilities — Rick Kittler
Part 3 – Local Roadway, Civil, Railroad Xing — Rick Kittler
Part 4 – Traffic — Pete Smith
Part 5 – Electrical — Mahmood Namazi
Part 6 – Environmental — Bill Jordan, Kate Snider
Part 7 – Casting Basin — Rick Kittler-Civil, Don Oates - Structures
Part 8 – Pontoons — Tom Schnetzer, Rick Johnson
Part 9 – Launch Channel — Don Oates
Part 10 – Mechanical — Don Oates
Part 11 & 12 – Geotechnical — Bob Mitchell

PART 2.0 BASIS OF DESIGN – SITE ROADWAY, CIVIL, UTILITIES

2.1 Roadway Tasks

2.1.1 Technical Tasks
2.1.2 Landscape
2.1.3 Agency Coordination

2.2 Design Requirements

2.2.1 Batch Plant

2.2.1.1. Layout - 2.82 acres site, with a paved process water containment area, water quality and TSS treatment system. The batch plant will be able to discharge into the west perimeter ditch and also have a separate infiltration pond as a contingency.

2.2.1.2. Loading –1,500 pounds per square foot (provided by batch plant supplier)

2.2.1.3. Operations – A site layout, truck path, and drainage plan will be developed by the batch plant supplier. The area will be graded t dran surface water to the ditch and north pond cells 1 and 2 and all process water will be contained.

2.2.2 Precast Yard

2.2.2.1. Layout

2.2.2.1.1. The precast Laydown area structural section will be designed according to heavy and light loading conditions. The section will be strengthened with geotextile for separation and geogrid.

2.2.2.1.2. The paved roadway from entrance to the main site access road will be designed according to the anticipated truck loads and construction vehicle traffic loads. From the main site access entrance and up into the parking lot will be designed according to passenger vehicle trips and loading.

2.2.2.2. Loading is designed for 25,000 pounds (WB67-truck). .

2.2.2.3. Operations– The precast yard will be concrete or asphalt for process water containment. The precast lay-down beds will be flat. Asphalt or concrete minimum thickness is 6-inches (without liners). The conveyance system from the precast lay-down beds will be designed according to the 25-year storm event (HRM 2008). The beds will be designed to contain flows in excess of the 25-year event.

2.2.3 Parking Facility

2.2.3.1. Cross section- The cross-section will be designed to drain into biofiltration swales for basic treatment prior to discharge. The structural section will be designed based on passenger vehicles and trip generation data. The section will be reinforced with construction geotextile for separation and geogrid for stabilization if warranted.

2.2.3.2. Parking Stalls- ~350 (18'x9') with 8 ADA van-accessible spaces (18'x9').

2.2.3.3. Drainage- shallow non-traversable biofiltration swales between rows (3:1 slopes; flat bottoms and grass lined for basic water quality treatment).

2.2.4 Casting Basin

2.2.4.1. Access Road- 35ft width for a WB67 and 16%. The vertical curve at the top of the basin is asphalt to high point and gravel north of high point. The casting basin roadway surface is asphalt.

2.2.5 Drainage

Highway Runoff Manual Best Management Practice RT 12. Wet Pond- Basic Water Treatment- linear relationship between basin area to volume required for pond. 3:1 side slopes, 1ft extra storage at bottom of pond (sediment cell) for sediment storage. The ponds shall have a 1-foot minimum freeboard, emergency outfall conditions for 100-year storm event, oil trapping control structures, and rubber check valve at outlet. Gate valves may be necessary in select structures for diverting stormwater for additional treatment, detention, or alternative discharge. The ponds are designed for the water quality treatment volume based on 91% of the 24 hour storm event runoff volume and surface area determined from sedimentation rates and the 10-yr flows.

2.2.5.1. Design Storm for pond sizing- The design was performed for the 91st percentile of the 24-hour event runoff volume and exceeds the square footage determined from the DOE 10-year peak flow settlement rate equation. The groundwater presettling pond will be designed according to the modeled pumping/draw-down rate for the casting basin construction.

2.2.5.2. The steady state flow (200 gpm) was determined through the geotechnical analysis and provided for the groundwater pump design. Conveyance will be determined according to the 25-year storm event. Emergency outfalls r conveyance systems will be designed according to the 100-year storm event. The groundwater collection system will be designed according to the steady state draw-down flows (200 gpm). Culverts will be designed according to the 10-year storm event. The minimum slope will be 0.2%, and meet minimum self-cleaning velocities.

2.2.5.3. Precast catch basin types 1 and 2 will be used and sized according to access, internal pipe angles, and pipe diameters.

2.2.5.4. No culvert will be installed under the proposed stockpile. Existing culverts crossed with heavy equipment will be protected with protection slabs designed according to the standard plan for bridge approaches.

2.2.6 Utilities – Water Distribution/Fire Suppression

2.2.6.1. Obtain final approvals from the authority having jurisdiction and local water purveyor.

2.2.6.2. Site Water: Site fire and domestic water supply is proposed to connect to City of Aberdeen water distribution system.

2.2.6.3. Estimated site fire flow demand is 2,000 gpm.

2.2.6.4. Water pipe from connection to existing 8-inch service lateral on site to downstream of fire and domestic water backflow preventers shall be ductile iron pipe conforming to AWWA C151, cement mortar lined, special thickness class 52, ranging from 2-inchto 10inch (laterals, mains), with mechanical joint or push on ends designed for use with a restraint system, installed according to AWWA C600 and AWWA M41. Pipe fittings shall be cement mortar lined ductile iron meeting the requirements of AWWA C110, C153 and C104 with joints meeting the requirements of AWWA C111. Pipe joints at tees and changes in direction anchored with restrained joints or thrust blocks.

2.2.6.5. Water pipe from downstream of site backflow preventers and throughout the site shall be shall be polyvinylchloride pipe.

2.2.6.5.1. Pipe ranging from 4-inch to 10-inch shall conform to AWWA C900 or C905 with minimum SDR of 18, UL listing and joints meeting ASTM D3139. Pipe fittings shall be cement mortar lined ductile iron meeting the requirements of AWWA C110, C153 and C104 with joints meeting AWWA C111. Pipe joints at tees and changes in direction shall be anchored with restrained joints or thrust blocks.

2.2.6.5.2. Pipe under 4-inches shall conform to ASTM D2241 with minimum wall thickness equal or greater than SDR 21, National Sanitation Foundation Seal for use to transport potable water and joints meeting ASTM D3139. Pipe fittings shall be polyvinylchloride and meet the requirements of ASTM D2466. Pipe joints at tees and changes in direction for 3-inch and larger pipe shall be anchored with thrust blocks.

2.2.6.6. Detector check and double check backflow preventer (BFP) conforming to AWWA C510 and isolation valves within an on-grade heated and insulated enclosure. The detector check shall be provided and installed by the City of Aberdeen.

2.2.6.7. Water meter and reduced pressure zone backflow preventer conforming to AWWA C511 and isolation valves within an on-grade heated and insulated enclosure..

2.2.6.8. Downstream of the site domestic water BFP, provide shut-off valves and reduced zone pressure BFP's serving project office, casting basins, and concrete batch plant. BFP's shall be within a below grade precast vault or otherwise freeze protected.

2.2.6.9. Fire suppression: Dry-barrel type fire hydrants conforming to AWWA C502, installed with separate gate valve in supply pipe, anchor with restrained joints or thrust blocks.

2.2.7 Utilities – Sanitary Sewer System

2.2.7.1.1. Site sanitary will gravity drain to City of Aberdeen sanitary collection system.

2.2.7.2. 6" PVC gravity sanitary sewer pipe conforming to ASTM D3034 and dimension ratio of SDR 35, ranging from 4"- 8" (laterals, mains) with water-tight push-on joints conforming to ASTM D3212 and sealing gasket conforming to ASTM F477.

2.3 Design Exceptions

2.4 Proposal Assumptions and Options Considered

2.4.1 Utilities – Water Supply

2.4.1.1. Water supply from nearby 12-inch city water main is assumed to be adequate.

2.4.2 Utilities – Sanitary Sewer

2.4.2.1. Site sanitary drains will gravity drain to city sanitary collection system.

2.5 Alternative Technical Concepts (ATCs)

2.5.1 ATC 10- Hydro seeded berm- reduces haul of unsuitable material

PART 3.0 BASIS OF DESIGN – LOCAL ROADWAY, CIVIL, RAILROAD CROSSING

3.1 Roadway Tasks

3.1.1 Technical Tasks

3.1.2 Agency Coordination

3.1.2.1. Local Streets- City of Aberdeen

3.1.2.2. Railroad- Puget Sound and Pacific Railroad (through WSDOT)

3.1.2.3. Utilities-

3.2 Design Requirements

3.2.1 ADA compliant

3.2.2 Turning movements, lane widths meet WSDOT Local Agency Guidelines standards

3.2.3 Design Vehicle- Tractor with 60' Stretch Trailer (variation of WB-67). In the casting basin, the vehicle was a low-boy trailer with 25,000 pounds per axle.

3.2.4 Roadway classifications- Principal Arterials, local collectors- Design Speed- 35mph (posted speed).

3.2.5 Signing- MUTCD 2009 manual

3.2.6 Existing City Streets, Sidewalks, and Driveways – meet existing sections for reconstruction/repairs.

3.3 Design Exceptions

None

3.4 Proposal Assumptions and Options Considered

3.4.1 Railroad Crossing- Gate signals to ameliorate lack of site distance

3.4.2 Signing- MUTCD compliant, used RFP signing

3.4.3 Drainage- minimal new pavement will be added (less than 2,000 SF in City ROW). Rehabilitation may involve possible mill & overlay depending on existing condition of pavement. No water quality, conveyance, or drainage modifications will be necessary for the off-site work elements.

PART 4.0 BASIS OF DESIGN – TRAFFIC

4.1 Project Elements

4.1.1 Signing

4.1.2 Striping –

4.1.3 Traffic Signals – no traffic signals anticipated

4.1.4 Rail Crossing Gate system – no gate warranted, PSAP will install new flashers

4.1.5 Public outreach- use noise-mitigating BMP's

4.2 Design Requirements

4.2.1 Traffic Analysis Plan
4.2.2 Traffic Management Plan
4.2.3 Traffic Incident Management Plan

4.3 Design Exceptions

None

4.4 Proposal Assumptions and Alternatives Considered

4.4.1 60% truck traffic reduction (69,300 trips) with smaller basin
4.4.2 Haul Route on established city streets with channelization near project site entrance
4.4.3 Used WSDOT traffic model from RFP
4.4.4 Traffic modeling will be conducted for design years 2011 thru 2014 with morning and evening peak hour periods.

PART 5.0 BASIS OF DESIGN – ELECTRICAL

5.1 Design Overview

5.1.1 The electrical design will support the construction facility to build and store pontoons. Major electrical distribution system equipment and components include:

5.1.1.1. Switchboards
5.1.1.2. Panelboards
5.1.1.3. Dry-Type Transformers
5.1.1.4. Safety Disconnect Switches
5.1.1.5. Stand-by Generator
5.1.1.6. Automatic Transfer Switches

5.2 Design Requirements

5.2.1 The electrical design will comply with all noted electrical Mandatory Standards.

5.2.1.1. Utility requirements

5.2.1.1.1. Largest provided utility transformer is 2500 kVA
5.2.1.1.2. Distribution at 480/277, 3 phase, 4 wire

5.2.1.1.3. Design to include one-line diagram, site plan with substation locations, motor data, distribution voltage, engineering contact and application for service

5.2.1.2. Power system study

5.2.1.2.1. Short-circuit calculations

5.2.1.2.2. Coordination curves

5.2.1.2.3. Arc flash calculations

5.2.1.2.4. Load flow calculations

5.2.1.2.5. Motor starting calculations

5.2.1.2.6. Harmonic analysis

5.2.1.3. Electrical Site Design

5.2.1.3.1. Sump Pumps (40 HP)

5.2.1.3.2. Valve Actuators (1/2 HP)

5.2.1.3.3. Construction equipment including Cranes, Trailers, and Lighting

5.2.1.3.4. Switchboards suitable to IBC seismic requirements

5.2.1.3.5. Switchboards and other indoor equipment provided in NEMA Type 1 enclosures

5.2.1.3.6. Electrical panels and other outdoor equipment provided in NEMA Type 4X enclosures

5.2.1.3.7. Motors larger than 40 HP shall be provided with soft starters

5.2.1.3.8. High efficiency exterior lighting shall be controlled by astronomical clock

PART 6.0 BASIS OF DESIGN – ENVIRONMENTAL

6.1 Design Overview

6.1.1 Environmental Compliance - All work will:

- avoid impacts beyond environmental documentation and permits
- Comply with laws and regulations

6.1.2 Mandatory Standards - Design and Construction will comply with WSDOT standards per RFP 2.8.2

WSDOT Standard Specifications and Plans

WSDOT Environmental Procedures Manual

WSDOT Environmental Compliance Assurance Procedure
WSDOT Highway Runoff Manual
WSDOT Design Manual
WSDOT Construction Manual
Ecology Stormwater Management Manual for W. Washington

6.1.3 WSDOT is responsible for NEPA environmental documentation. K-G Team support and provide design and construction information as requested. ROD will identify project alternative selected for Final Design. Final Design or Construction shall not begin until ROD is issued by FHWA

6.1.4 Design and Construction will meet all requirements defined in RFP Commitments List and any other requirements per final NEPA, permits and approvals. Update Commitments List as project progresses and additional permits and approvals are obtained by WSDOT and K-G. Updated commitments list submitted with Final Design

6.1.5 WSDOT responsible for JARPA Permits, including:

Section 404/Section 10 – Corps of Engineers
401 Water Quality Certification – Dept. of Ecology
HPA – WDFW
CZMA – ecology
Aquatic Lands Use Authorization – DNR
Shoreline Substantial Development – Local Jurisdictions
Critical Areas Compliance – Local Jurisdictions

K-G Team support and provide design and construction information as requested. JARPA submittal K-G to WSDOT to include items per 2.8.5.1.

6.1.6 K-G responsible the following Permits and all other necessary permits not identified

PATON – US Coast Guard
NPDES Construction Stormwater General – Dept. of Ecology
NPDES Sand and Gravel General – Dept. of Ecology
Disposal Site Use Authorization – WDNR
Air Quality Notice of Construction for batch plant – ORCAA
Notices of Intent for borings and wells – Dept. of Ecology
Street Use Permit – Aberdeen
Noise Variance – Aberdeen (determined not necessary)
Building Permit – Aberdeen
NPDES Municipal Permit – Aberdeen (will be covered under the WSDOT/DOE NPDES Sand and Gravel Permit).
Administrative Order for Chemical Treatment - Ecology

Direct interactions with permit agencies

WSDOT informed and invited to participate – consistency

6.1.7 K-G Environmental Compliance Plans

Roles and responsibilities, communications protocol, procedures to ensure environmental compliance, correct noncompliance, emergency response

Consistent with WSDOT and K-G Requirements

Consistent with permit requirements

Environmental Compliance Plan (ECP) to include the following components:

Personnel, Communications and Training Protocols

Temporary Erosion and Sediment Control Plan (TESC)

Spill Prevention Control and Countermeasure Plan (SPCC)

Roadside Work Plan

Fugitive Dust Control Plan

Unanticipated Archaeological Discovery Plan (WSDOT author)

Monitoring Plan

Soil Management Plan

Collection Containment and Disposal Plan

Final ECP to be finalized following permit issuance to incorporate final permit conditions

6.1.8 K-G required associated plans

Additional Plans to be submitted separate from the ECP but prior to construction start include:

Water Quality Monitoring and Protection Plan for In-Water Work

Dissolved Oxygen Monitoring and Contingency Plan (part of WQMPP)

Quality Control Plan for Dredging and Disposal

Monitoring Plan for Underwater Sound

Restoration Plan for Temporary Shoreline Impacts

Worker Health and Safety Plan

Concrete Batch Plant O&M Manual

Casting Facilities Operations Manual

6.2 Design Requirements

6.2.1 Mandatory Standards - Design and Construction will comply with WSDOT standards per RFP 2.8.2

WSDOT Standard Specifications and Plans

WSDOT Environmental Procedures Manual

WSDOT Environmental Compliance Assurance Procedure
WSDOT Highway Runoff Manual
WSDOT Design Manual
WSDOT Construction Manual
Ecology Stormwater Management Manual for W. Washington

6.2.2 Design and Construction will meet all requirements defined in RFP Commitments List and any other requirements per final NEPA, permits and approvals.

6.3 Proposal Assumptions and Options Considered

Design does not trigger need for additional environmental review beyond WSDOT EIS. All work will be conducted within WSDOT DEIS Impact Area Line

PART 7.0 BASIS OF DESIGN – CASTING BASIN

7.1 Design Overview

7.1.1 Basin Functional Requirements

7.1.1.1. Provide a facility capable of constructing (in the dry) and launching, a minimum of four (4) standard pontoons and two (2) supplemental pontoons per casting cycle.

7.1.1.2. The casting basin shall be designed to prevent catastrophic collapse of the basin structure and all structures contained within the basin, and or, sudden flooding.

7.1.1.3. The casting basin shall include a flooding and emptying system per Part 11.0

7.1.1.4. The minimum service life of the corrosion protection system for the steel elements shall be 15 years.

7.1.1.5. The maximum elevation of the top of the casting basin slab shall be (-9.0) feet.

7.1.1.6. A fish collection system shall be provided.

7.1.2 Casting Basin Description

7.1.2.1. Overview

The casting basin includes the following structural and functional elements: an excavated basin with a pile supported reinforced concrete slab and fish collection system, concrete bulkhead walls at the entrance, a pile supported reinforced concrete sill and pile supported reinforced concrete jambs at the entrance gate, flood control and sump structures, two pile supported tower gantry crane support structures, a pile supported gate setting and removal trestle, two pile supported temporary pre-cast staging trestles, a line of continuous fenders on the west side of the basin and a line of discontinuous fenders on the east side of the basin.

7.1.2.2. Horizontal Layout

The casting slab footprint is approximately 940 feet by 185 feet, which allows for a pontoon side clearance of 10 feet, and a clearance between pontoons of 15 feet. The casting slab has a 0% slope both transverse and longitudinal. The clear width between the two internal fender system lines is 182 feet.

The basin side slope cut is 2.5:1 from the top of a toe wall to a top elevation of +17.0 feet MLLW on the east side of the basin and +18.0 feet MLLW on the west side of the basin.

The depth of the basin allows for launching of pontoons having a saltwater draft of 17.0 feet plus a 1'-0" clearance to the top of the casting slab. Using these constraints, the minimum launch tide shall be +9.0 feet MLLW for pontoons other than Type 1 and Type 1A.

7.1.2.3. Vertical Layout

7.1.2.3.1. Tidal Elevations

The following tides are taken from NOAA Station #9441187, Aberdeen, WA and are presented here using a datum of MLLW = 0.0'

- Highest Observed Tide (HOT) +13.86 feet
- Mean Higher High Water (MHHW) +10.11 feet
- Mean High Water (MHW) +9.41 feet
- Mean Tide Level (MTL) +5.60 feet
- Mean Sea Level (MSL) +5.44 feet
- Mean Low Water (MLW) +1.47 feet

- Mean Lower Low Water (MLLW) +0.00 feet
- Lowest Observed Tide -3.35 feet

7.1.2.3.2. Casting Basin Elevations

The following elevations are based on MLLW = 0.0 feet

- Top of Basin East Side +17.00 feet
- Top of Basin West Side +18.00 feet
- Top of casting slab/fish collection -9.00 feet
- Top of gate rest logs -9.00 feet
- Top of sill -9.00 feet
- Top of sill at rest logs -9.50 feet
- Top of Sill at gate notch -11.00 feet
- Top of Bulkhead walls +18.00 feet
- Top of jambs +18.00 feet
- Top of gantry crane support structures East side +17.00 feet
- Top of gantry crane support structures West side +18.00 feet
- Bottom of entrance channel -13.00 feet

7.1.3 Design Codes and Specifications

1. WSDOT Standard Specifications and Amendments, (M41-10), 2008, Washington State Department of Transportation.
2. WSDOT Bridge Design Manual LRFD (M23-50), May 2008, Washington State Department of Transportation.
3. WSDOT Geotechnical Design Manual (GDM), Revised Chapter 6 and Chapter 8, August 14, 2009, Washington State Department of Transportation.
4. WSDOT Geotechnical Design Manual (M46-03), November 2008, Washington State Department of Transportation.
5. WSDOT Design Manual (M22-01), June 2009, Washington State Department of Transportation.
6. 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design, American Association of State Highway and Transportation Officials.

7. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, Fourth Edition (2007), with 2009 Interim Revisions, American Association of State Highway and Transportation Officials.
8. IBC International Building Code, (2006), International Building Code Council, Inc., as modified by the Washington State Building Code Council.
9. Minimum Design Loads for Buildings and Other Structures, ASCE / SEI 7-05, American Society of Civil Engineers.
10. AISC Manual of Steel Construction, 13th Edition, (2005), American Institute of Steel Construction (as a reference or as allowed by AASHTO).
11. ACI Building Code Requirements for Structural Concrete with Commentary, ACI 318-05, American Concrete Institute.
12. ACI Suggested Analysis and Design Procedures for Combined Footings and Mats, ACI 336.2R-88(2002), American Concrete Institute.
13. Guide for the Design and Construction of Fixed Offshore Concrete Structures, ACI 357R-84 (Reapproved 1997), American Concrete Institute.
14. U.S. Department of Defense, UFC 4-213-10, "Design: Graving Docks".
15. Washington Industrial Safety and Health Act (WISHA), Guidelines for Platforms, Ladders and Railings.

7.1.4 Reference Documents

1. SR 520 Pontoon Construction Design-Build Project, Chapter 2, Technical Requirements, Request for Proposal, August 24, 2009.
2. <u>SR 520 Pontoon Casting Facility Coastal Engineering Report, Coast and Harbor Engineers, January 14, 2010</u>
3. <u>SR 520 Pontoon Casting Facility Geotechnical Report, Shannon and Wilson, January 14, 2011</u>
4. Design of Marine Facilities for the Berthing Mooring, and Repair of Vessels Second Edition, Gaythwaite, John W. ASCE Press, American Society of Civil Engineers.

7.1.5 Loads

7.1.5.1. Material Densities

- Reinforced Concrete 160 pcf
- Seawater 64 pcf
- Steel (including stainless) 490 pcf

7.1.6 Load Combinations

Load factors and combinations considered in the design are based on Table 3.4.1-1 from the AASHTO LRFD Bridge Design Specifications, unless noted otherwise.

- AASHTO Strength I
- AASHTO Strength II
- AASHTO Strength III
- AASHTO Strength IV
- AASHTO Strength V
- AASHTO Extreme Event I
- AASHTO Extreme Event II
- AASHTO Service I
- AASHTO Service II

7.1.7 Materials

7.1.7.1. Concrete

- Cast-in-place concrete shall be Class 4000 or greater.

7.1.7.2. Non-Shrink Cementitious Grout

- Non-Shrink Cementitious Grout shall conform to ASTM C 1107
- Compressive Strength (ASTM C 109) 10,000 psi
- Early Height Change (ASTM C 827) 0.0 to 4.0%
- Hardened Height Change (ASTM C 1090) 0.0 to 0.3%
- Effective Bearing Area, 95%

7.1.7.3. Mild Reinforcing Steel

- Reinforcing steel shall be ASTM A706 Grade 60 unless otherwise noted.
- Prestressing strand shall be AASHTO M203 (ASTM A416) B Grade 270, Low Relaxation
- Smooth spiral wire shall be ASTM A82

7.1.7.3.1. Splice and Development Length per WSDOT BDM

7.1.7.3.2. Concrete Cover (primary reinforcement, stirrups ½" less) shall be:

- 4" for surfaces permanently exposed to saltwater

- 3" at all other locations UNO

7.1.7.3.3. Exposure Factor γ_e (AASHTO 5.7.3.4)

- 0.75 for surfaces permanently exposed to saltwater
- 1.00 for all other surfaces

7.1.7.3.4. Bar lists to be provided by supplier

7.1.7.4. Structural Carbon Steel

Unless noted otherwise:

- W and HP shapes shall be AASHTO M 270 Grade 50
- M, MC, C, S and L shapes shall be AASHTO M270 Grade 36 or AASHTO M270 Grade 50 where required
- Plate shall be AASHTO M260 Grade 36 or AASHTO M270 Grade 50 where required
- Rectangular HSS shall be ASTM A500 Grade B, Fy = 46ksi
- Round HSS shall be ASTM A500 Grade B, Fy = 42ksi
- Pipe shall be ASTM A53 Grade B, Fy = 35ksi (unless noted otherwise)
- Fasteners shall be High Strength ASTM A325,galvanized
- Friction type connections: Class B coating on faying surfaces

7.1.7.5. Stainless Steel

- Stainless steel plate shall be ASTM A240, Type 316L
- Stainless steel shims shall be ASTM A240, Type 316

7.1.7.6. Piling

- Steel Pipe Piles ASTM A252 GR 2, except that the yield strength shall be a minimum of 45, 50, or 55 ksi as required by the Drawings and with additional requirements per Specifications

7.1.7.7. Ultra High Molecular Weight Polyethylene (UHMW)

- ASTM D4020

7.1.7.8. Corrosion Protection

- Structural steel elements, connections, fittings, etc. shall have corrosion protection for a lifespan of 15 years in a marine environment.

- Steel piling will be coated from the top of the pile to 25' below the lowest adjacent ground level.
- Structures and equipment in or adjacent to the launch channel shall be of Type 316L stainless steel or equally corrosion resistant material, except for guide piles, dolphin pilings, structural shapes and plates used to construct guide and turning dolphins.

7.2 Pile Supported Casting Slab

7.2.1 Pile Supported Casting Slab Functional Requirements

The casting basin slab is a fixed, pile supported structure where pontoons are cast. The slab is designed to allow for multiple pontoons to be constructed and launched. The slab shall incorporate a system to collect and transport fish so that the fish can be returned to the channel after the gate is closed and prior to completely draining the casting basin.

7.2.2 Casting Slab Description

7.2.2.1. Overview

The casting slab is a reinforced concrete slab structure supported on steel pipe piles. Circular drop caps are provided at each pile connection to provide additional punching shear capacity so that a thinner slab may be utilized for slab bending resistance.

7.2.2.2. Horizontal Layout

- The casting basin width from face of toe wall to face of toe wall is 185'.
- The casting basin length from back of basin slab to is the front face of the jambs is 939' – 10".

7.2.2.3. Vertical Layout

The following elevations are based on MLLW = 0.0 feet

- Top of casting slab -9.00 feet
- Top of side slope toe wall -5.00 feet

7.2.3 Design Approach

The casting basin piles shall be designed to carry the vertical loads imparted from and to the casting basin slab in addition to providing the primary lateral shear support for the gate, sill, jamb and bulkhead wall structures under hydrostatic and seismic loading conditions.

7.2.4 Design Codes and Specifications

- See Section 7.13

7.2.5 Loads

7.2.5.1. Dead Load, DC

- Self-weight (see Section 7.1.5.1 for material densities)

7.2.5.2. Pontoon Weight, PN

Pontoons have the same load factors and appear in the same combinations as that of dead load due to non-structural components and non-structural attachments (DC), except that the maximum load factor for the STRENGTH IV case shall be 1.35.

The basin floor shall be designed for the weights of the pontoons to be built, and shall incorporate pattern loading to determine the maximum effects of the pontoon on the basin floor. Design Pontoon loads are as follows:

  - A uniform load of 1000 psf over 60 foot by 100 foot footprint with 20 feet -0 inches minimum clear spacing between each 60 foot by 100 foot footprint.
  - A uniform load of 1150 psf over a 50 foot by 100 foot footprint contained within each 60 foot by 100 foot footprint described above.
- A uniform load of 1010 psf over a 60 foot by 100 foot footprint with 20 feet -0 inches minimum clear spacing between each 60 foot by 100 foot footprint.
- A uniform load of 1150 psf over the entire floor.

7.2.5.3. Live Loads

7.2.5.3.1. Construction live load of 40 psf on the pontoons

7.2.5.3.2. Pontoon formwork and bracing loads

7.2.5.3.3. HL-93 Vehicular live load

7.2.5.3.4. Pedestrian live load of 85psf over the basin floor

7.2.5.4. Uniform Temperature, TU

The temperature range shall be 0 to 100 with a starting temperature of 64 degrees (36 degrees up and 64 degrees down).

7.2.5.5. Seismic Loading, EQ

The basin shall be designed to withstand the 1,000 year earthquake defined by the project site-specific response spectrum. Minimum response shall not be taken less than 2/3 of the code based response spectrum.

- The horizontal seismic loads shall include accelerations from 50% of the mass of the Pontoons acting in combination with 100% of the vertical weight of the Pontoons.
- Earthquake Resisting Elements (ERE)
  The following item shall be designated as the ERE:
    - Casting Basin piles with "Pinned Head"
    - Piles shall be design to remain "essentially elastic" during seismic event

7.2.5.6. Downdrag, DD

- There are no downdrag loads on the piles under static load conditions.
- Liquefaction induced downdrag loads shall be applied to the piles in only the post-earthquake loading condition.

7.2.5.7. Horizontal Earth Pressure, EH

See geotechnical report for horizontal earth pressures applied to basin toe-wall

7.2.6 Slab Deflection Criteria

The slab deflection shall be limited such that the following specified Pontoon fabrication tolerances are maintained:

- Pontoon slab thickness shall be plus 1/8 inch, minus zero inches
- The skew of the Pontoon across the end section shall be a maximum of 1/2 inch

To meet the above specified Pontoon fabrication tolerances the slab shall be designed to meet the following deflection criteria:

- Max allowable slab deflection between piles, 1/8 inch
- Max allowable differential settlement across the width of a Pontoon, 1/2 inch

7.2.7 Load Combinations

See Table 7.2.7 for a compilation of the loads and load combinations used for design of the slab.

Table 7.2.7

Load Combination Limit State	DC	PN	DD	EH	LL	WA		TU (3)	TG	EQ
						HL (1)	WL (2)			
Strength I	1.25	1.25	--	1.50	1.75	1.00	--	0.5	--	--
Strength I	1.25	1.25	--	1.50	1.75	--	1.00	0.5	--	--
Strength II	1.25	1.25	--	1.50	1.35	1.00	--	0.5	--	--
Strength II	1.25	1.25	--	1.50	1.35	--	1.00	0.5	--	--
Strength III	1.25	1.25	--	1.50	--	1.00	--	0.5	--	--
Strength III	1.25	1.25	--	1.50	--	--	1.00	0.5	--	--
Strength IV	1.50	1.35	--	1.50	--	1.00	--	0.5	--	--
Strength IV	1.50	1.35	--	1.50	--	--	1.00	0.5	--	--
Extreme I (4), (5)	1.00	1.00	--	1.50	--	1.00	--	--	--	1.00
Extreme I (4), (6)	1.00	1.00	1.25	1.50	--	1.00	--	--	--	1.00
Service I	1.00	1.00	--	1.00	1.00	1.00	--	1.00	0.50	--
Service I	1.00	1.00	--	1.00	1.00	--	1.00	1.00	0.50	--
Service II	1.00	1.00	--	1.00	1.30	1.00	---	1.00	--	--
Service II	1.00	1.00	--	1.00	1.30	--	1.00	1.00	--	--
Service III	1.00	1.00	--	1.00	0.80	1.00	--	1.00	0.50	--
Service III	1.00	1.00	--	1.00	0.80	--	1.00	1.00	0.50	--

Service IV Service IV	1.00 1.00	1.00 1.00	-- --	1.00 1.00	-- --	1.00 --	-- 1.00	1.00 1.00	1.00 1.00	-- --

DC = Dead load of structural components
LL = Vehicular and uniform live load
HL = Hydrostatic load from Gate & Bulkhead (2)
PLL = Pedestrian and work platform loads
TG = Temperature Gradient

PN = Pontoon Load
WA = Static water loads
EQ = Seismic Force
TU = Uniform Temperature
EH = Horizontal Earth Pressure

Notes:

(1) Hydrostatic shear from Gate & Bulkhead with tide at MHHW
(2) Uniform water pressure on slab with tide at MHHW
(3) If considering deflections, TU load factor shall be taken as 1.20
(4) Per section 3.7 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design a load factor of 1.0 is used on dead and pontoon loads. Vertical PN loads are calculated using full weight of pontoons.
(5) EQ loads are the inertial loads calculated using full weight of structure and half the mass of the pontoons.
(6) EQ loads are the based on the imposed displacements from FLAC analysis.

7.3 Casting Basin Bulkhead Wall

7.3.1 Casting Basin Bulkhead Wall Functional Requirements

A concrete and sheet pile bulkhead wall runs along the south edge of the casting basin extending westward and eastward from the jambs, sill and gate structure. The ends of the bulkhead wall will transition from the cast-in-place concrete structure to a steel sheet pile wall which terminates in the upland soils of the casting yard. The wall is designed to prevent water in the launch channel from entering the basin when the gate structure is in place.

7.3.2 Bulkhead Wall Description

7.3.2.1. Overview

The casting basin bulkhead wall consists of a cast-in-place concrete wall and foundation. The concrete bulkhead wall is three feet thick and has a walkway along the top, allowing construction personnel to cross the bulkhead walls, jambs and gate structure from one side of the casting yard to the other. A steel sheet pile cutoff wall is driven below the bottom of the wall foundations to restrict infiltration of water into the casting basin. The east and west ends of the wall are terminated with a steel sheet pile wall. The sheet pile wall at the both ends is capped with a wide flange beam laid flat to extend the walkway on the top.

7.3.2.2. Horizontal Layout

The bulkhead wall is parallel to Grid A of the gate structure. The channel face of the wall is 2 foot 6 inches north of the jamb control points. The east jamb bulkhead wall structure (Wall E1) is extended an additional 45 feet eastward for an overall length of 68 feet. This portion of the bulkhead wall will be supported laterally by the casting basin slab, similar to the gate and sill structure. A 4 feet 5 inch section of sheet pile wall is located between the jamb bulkhead walls and the bulkhead walls to allow for differential seismic movement between the structures (seismic joint). The bulkhead walls extend 63 feet eastward (27 foot concrete and 36 foot sheet pile) and 83 feet westward (59 foot concrete and 24 foot sheet pile) of the seismic joints at the ends of the jamb structures.

7.3.2.3. Vertical Layout

7.3.2.3.1. Bulkhead Wall Elevations

The following elevations are based on MLLW = 0.0 feet

- Top of wall + 18.00 feet
- Top of sill - 9.00 feet
- Top of Casting Basin - 9.00 feet

The concrete bulkhead wall footings are stepped upward in various increments as shown on the drawings.

7.3.2.4. Design Approach Concrete Bulkhead Walls

7.3.2.4.1. Load Path

- Vertical and overturning loads are resisted by vertical piles under the bulkhead wall foundations.
- Lateral loads (hydrostatic, earth, live load surcharge, seismic, and wind) towards the basin or channel on the bulkhead walls are resisted by cantilever action of the wall and by vertical piles under the bulkhead wall foundations. Elastomeric steel bearing pads at the top

of the bulkhead wall limit the transfer of loads between the west bulkhead wall and trestle. The wall is designed assuming the bearings due not fuse during a seismic event.

- Earthquake Resisting Elements (ERE)

The following item shall be designated as the ERE:

- Bulkhead Piles with in-ground hinging

7.3.2.5. Design Approach Sheet Pile Walls

7.3.2.5.1. Load Path

- Lateral loads (hydrostatic, earth, live load surcharge, seismic, and wind) towards the basin or channel on the sheet pile walls are resisted by bending between the wide flange brace at the top and the point of fixity in the ground.

7.3.3 Bulkhead Trestle Description

7.3.3.1. Overview

The bulkhead trestle is located on and to the north of the west bulkhead wall. The bulkhead is designed to provide an access platform for tracked and mobile truck cranes to remove and reset the three gate sections. The trestle is 48 feet 0 inches wide in the east/west direction and 53 feet 6 inches wide in the north/south direction. The trestle structure consists of 6 foot wide haunched-precast-prestressed concrete deck panels. The deck panels are supported by three pile bents on the north side and near the middle of the trestle. The panels on the south side of the trestle are supported by the precast concrete bulkhead walls. The trestle structure at the bulkhead wall line is set on bearing pads which allow for differential horizontal and rotational movement between the trestle structure and the supporting jamb and bulkhead walls.

7.3.3.2. Design Approach Trestle

7.3.3.2.1. Load Path

- Vertical loads are supported by the deck panels, which transfer loads to the pile caps and the bulkhead walls. The pile caps and bulkhead wall transfer loads to vertical support piles.

- Lateral loads on the trestle are resisted by the vertical piles. Elastomeric steel bearing pads at the top of the bulkhead wall transfer a portion of the strength and service loads to the bulkhead wall. The trestle is designed assuming the bearings fuse during a seismic event.

The following item shall be designated as the ERE:

- Bulkhead Trestle Piles with in-ground hinging

7.3.4 Design Codes and Specifications

- See Section 7.1.3

7.3.5 Loads

7.3.5.1. Dead Load, DC

- Self-weight (see Section 7.1.5.1 for material densities)

7.3.5.2. Live, PL

- Pedestrian Live Load of 85 psf along the top walkway.

7.3.5.3. Crane Loads, CR

- The trestle shall be designed to support the following cranes to allow for the removal of the gate structure: Manitowoc 2250 crawler crane, GMK 7550 truck crane, or a Demag 1200 TC truck crane.

7.3.5.4. Horizontal Earth Pressure, EH

- See Geotechnical Report

7.3.5.5. Vertical Earth Pressure, EV

- See Geotechnical Report

7.3.5.6. Live Load Surcharge

- Mobile truck crane at 2.55 klf with the nearest wheel line at a distance of 15 feet from the back face of the wall. Lateral wall pressures shall be in accordance with Figure 31 of the Geotechnical report.
- The Manitowoc 2250 Crawler crane (or any vehicle that exceeds the mobile truck crane loading described in this section) shall not to be driven within 60 feet the bulkhead walls.

7.3.5.7. Tide, WA

- The bulkhead wall shall be designed to resist the forces due to water pressures caused by tidal fluctuations. Highest observed tide is +13.86 feet

MLLW per the coastal engineering report. Buoyant forces are only present on the channel side of the sheet pile cutoff wall.

7.3.5.8. Flood, WA

- The bulkhead wall shall be designed to withstand the FEMA 50 year flood as defined in the coastal engineering report at an elevation of +14.9 feet MLLW.

7.3.5.9. Wave, WV

- Wave loading assumes a 50 year return period, and waves out of the south. Wave heights and periods from the south are 1.8 feet and 1.9 seconds per the coastal engineering report.

7.3.5.10. Wind, WS

- Wind loading on the bulkhead wall assumes a base wind speed of 105 mph as recommended in the coastal engineering report.

7.3.5.11. Seismic, EQ

- The bulkhead walls shall be designed to withstand the 1,000 year earthquake defined by the project site-specific response spectrum. Minimum response shall not be less than 2/3 of the code based response spectrum.
- Seismic component of horizontal earth pressure:
  9xH psf at top of the ground on the high side
  2xH psf at top of the ground on the low side
  Where H = Difference in soil height from the top of the ground on the high side to the top of the ground on the low side.

7.3.5.12. Downdrag, DD

- There are no downdrag loads on the piles for the bulkhead wall and trestle per the Geotechnical report.

7.3.5.13. Temperature, TU

- The temperature range shall be 33 degrees up and 33 degrees down.

7.3.6 Load Combinations

See Table 7.3.6 for a compilation of the loads and load combinations used for design of the bulkhead walls and trestle.

Table 7.3.6

Load Combinations and Load Factors

Load Combination Limit State	DC	LS BR(7)	CR (1)	EH (2)	EV	WA (3) FEMA Flood	WA (3) MHHW	WA (3) LOT	WV	WS	EQ	TU(4)
Strength I	1.25	1.75	1.36	γ_p	1.35	--	--	0.90	--	--	--	0.5
Strength I	0.90	1.75	--	γ_p	1.00	--	--	0.90	--	--	--	0.5
Strength II	1.25	--	1.36	γ_p	1.35	--	1.00	--	--	--	--	0.5
Strength II	0.90	--	1.36	γ_p	1.00	--	1.00	--	--	--	--	0.5
Strength III	1.25	--	1.36	γ_p	1.35	1.00	--	--	1.40	1.40	--	0.5
Strength III	0.90	--	--	γ_p	1.00	1.00	--	--	1.40	1.40	--	0.5
Strength IV	1.50	1.75	1.36	γ_p	1.35	--	1.20 (5)	--	--	--	--	0.5
Extreme I	1.25	--	--	γ_p	1.35	--	1.00	--	--	--	1.00	0.5
Extreme I	0.90	--	--	γ_p	1.00	--	1.00	--	--	--	1.00	0.5
Extreme I	1.25	--	--	γ_p	1.35	--	--	0.90	--	--	1.00	0.5
Extreme I	0.90	--	--	γ_p	1.00	--	--	0.90	--	--	1.00	0.5
Service I	1.00	1.00	1.00	1.00	1.00	--	--	1.00	--	--	--	1.00
Service II	1.00	1.00	--	1.00	1.00	--	--	1.00	--	--	--	--
Service III	1.00	--	1.00	1.00	1.00	--	1.00	--	1.40 (6)	--	--	1.00
Service IV	1.00	--	--	1.00	1.00	--	1.00	--	1.40 (6)	1.40 (6)	--	1.00

Notes:
(1) The crane load factor is based on 1.25 x crane dead load in combination with 1.25 impact x 1.75 x 90 kip gate load.

(2) γ_p for EH is 1.5 and 0.9.
(3) Includes buoyancy.
(4) If considering deflections, TU load factor shall be taken as 1.2
(5) Assigns a higher load factor to account for impact of tides above MHHW
(6) Use the two year wave and wind condition for service loads.
(7) Breaking loads for the crane are based on a 10% x 680 kip crane weight = 68 kips. This equates to a stopping distance of about 2 inches at a maximum speed of /mph

DC = Dead load of structural components
LS = Vehicular Live Surcharge Load
CR = Crane Load
EH = Earth Pressure Horizontal
BR = Manitowoc 2250 Crane Breaking
WA = Static water loads
WV = Wave force
WS = Wind force
EQ = Seismic force

7.4 Pile Supported Sill and Jambs

7.4.1 Pile Supported Sill and Jambs Functional Requirements

7.4.1.1. Provide vertical and horizontal support for the stop log gate structure.

7.4.1.2. Provide a 110 foot clear opening between jambs.

7.4.1.3. The jamb and sill structure shall be designed to resist catastrophic collapse and sudden flooding of the casting basin.

7.4.2 Pile Supported Sill and Jambs Description

7.4.2.1. Overview

The sill and jamb structure is a reinforced concrete structure supported on steel pipe piles. The jamb foundations are approximately 24 feet x 30 feet. The sill foundation is approximately 25 feet x 96 feet between the two jamb foundations. The top portion of the jamb foundations will be cast integral with the sill foundation and a portion of the casting basin slab. The jambs are 8 feet by 12 feet and project 27 feet above the top of sill elevation. Each jamb is notched at one corner to accommodate seating of the gate structure.

A 23 foot long bulkhead wall extends eastward from the east jamb plus an additional 45 feet of Wall E1 for an overall length of 68 feet. Two large diameter pipes for flooding the casting basin pass through the bulkhead wall. This wall will be cast integral with the jamb.

A 9 foot long wall extends westward from the west jamb. This wall will be cast integral with the jamb.

7.4.2.2. Horizontal Layout

The jambs are designed to provide a 110 foot wide clear opening for removal of pontoons from the casting yard with the stop log gate removed.

7.4.2.3. Vertical Layout

- Sill and Jamb Elevations

The following elevations are based on MLLW = 0.0 feet

- Top of Jambs +18.00 feet
- Top of Sill -9.00 feet

7.4.2.4. Construction Tolerances

- See Section 8.1.10 Gate Fit Up for construction tolerances at the Gate/Sill and Jamb interfaces.

7.4.2.5. Design Approach

- Lateral Load Path (Towards Basin or Channel)
  - Gate loads (hydrostatic, seismic, and wind) are reacted through bearing points located along the vertical face of the jambs and running along the vertical face of the outer sill edge beam
    - Overturning loads at each jamb are resisted by the pile group under the jamb foundation.
    - Shear loads at each jamb and along the sill are resisted by the casting basin slab piles (sill and jamb foundation piles inclusive).
    - At the bulkhead wall, overturning loads are resisted by the piles directly below the wall. Shear loads from the wall are resisted by the casting basin slab piles.
- Vertical Load Path

  Gate loads are transferred to the sill foundation at 4 discreet bearing points located at the intersections of gate grids 3 and 7 and gate grids A and B. These loads are reacted by the sill foundation piles.

- Thermal Movements
  The sill and jamb piles are detailed to accommodate global thermal movement experienced by the casting basin slab.

7.4.3 Design Codes and Specifications

- See Section 7.1.3

7.4.4 Loads

7.4.4.1. Dead Load, DL

- Self-weight (see Section 7.1.5.1 for material densities)
- Gate dead load

7.4.4.2. Live, LL

- Vehicle live load, AASHTO HL-93

7.4.4.3. Live, PL

- Platform live load of 85 psf

7.4.4.4. Pontoon, PN

7.4.4.4.1. A uniform load of 1150 psf over the entire floor.

7.4.4.5. Tide, WA

- The sill and jambs shall be designed to resist the forces due to water pressures caused by tide fluctuations on the sill, jamb and gate structure. Highest observed tide is +13.86 feet MLLW per the coastal engineering report. Buoyant force effects are only present on the channel side of the sheet pile cutoff wall.
- The sill shall be designed for the basin flooded at MHHW, +10.11 feet MLLW with no balancing uplift on the basin side of the cutoff wall.

7.4.4.6. Flood, WA

- The sill and jamb shall be designed to withstand the FEMA 50 year flood as defined in the coastal engineering report as an elevation of +14.9 feet MLLW.

7.4.4.7. Horizontal Earth Pressure, EH

- See Geotechnical Report

7.4.4.8. Vertical Earth Pressure, EV

- See Geotechnical Report

7.4.4.9. Wave, WV

- Wave loading assumes a 50 year return period, and waves out of the south. Wave heights and periods from the south are 1.8 feet and 1.9 seconds per the coastal engineering report

7.4.4.10. Wind, WS

- Wind loading on the sill and jambs assumes a base wind speed of 105 mph as recommended in the coastal engineering report.

7.4.4.11. Seismic, EQ

- The basin shall be designed to withstand the 1,000 year earthquake defined by the project site-specific response spectrum. Minimum response shall not be taken less than 2/3 of the code based response spectrum.
- Earthquake Resisting Elements (ERE)
  The following item shall be designated as the ERE:
    - Casting Basin piles with "Pinned Head"

7.4.4.12. Downdrag, DD

- There are no downdrag loads on the piles for the sill and jamb structures per Geotechnical report.

7.4.4.13. Temperature, TU

- The temperature range shall be 33 degrees up and 33 degrees down.

7.4.5 Load Combinations

See Table 7.4.5 for a compilation of the loads and load combinations used for design of the sill and jambs.

Table 7.4.5

Load Combinations and Load Factors											
Load Combination Limit State	DC	LL & PLL(1)	EH	EV	WA (2) FEMA Flood	WA (2) MHHW	WA (2) LOT	WV	WS	EQ	TU(3)
Strength I	1.25	1.75	1.50	1.35	1.00	--	--	--	--	--	0.5
Strength I	0.90	1.75	1.50	1.00	1.00	--	--	--	--	--	0.5
Strength I	1.25	1.75	1.50	1.35	--	1.20 (4)	--	--	--	--	0.5
Strength I	0.90	1.75	1.50	1.00	--	1.20 (4)	--	--	--	--	0.5
Strength II	1.25	1.35	1.50	1.35	1.00	--	--	--	--	--	0.5
Strength II	0.90	1.35	1.50	1.00	1.00	--	--	--	--	--	0.5
Strength II	1.25	1.35	1.50	1.35	--	1.20 (4)	--	--	--	--	0.5
Strength II	0.90	1.35	1.50	1.00	--	1.20 (4)	--	--	--	--	0.5
Strength III	1.25	--	1.50	1.35	1.00	--	--	1.40	1.40	--	0.5
Strength III	0.90	--	1.50	1.00	1.00	--	--	1.40	1.40	--	0.5
Strength III	1.25	--	1.50	1.35	--	1.20 (4)	--	1.40	1.40	--	0.5
Strength III	0.90	--	1.50	1.00	--	1.20 (4)	--	1.40	1.40	--	0.5
Strength IV	1.50	--	1.50	1.35	1.00	--	--	--	--	--	0.5
Strength IV	0.90	--	1.50	1.00	1.00	--	--	--	--	--	0.5
Strength IV	1.50	--	1.50	1.35	--	1.20 (4)	--	--	--	--	0.5
Strength IV	0.90	--	1.50	1.00	--	1.20 (4)	--	--	--	--	0.5
Strength V	1.25	1.35	1.50	1.35	1.00	--	--	0.40	0.40	--	--
Strength V	0.90	1.35	1.50	1.00	1.00	--	--	0.40	0.40	--	--
Strength V	1.25	1.35	1.50	1.35	--	1.20 (4)	--	0.40	0.40	--	--
Strength V	0.90	1.35	1.50	1.00	--	1.20 (4)	--	0.40	0.40	--	--

Extreme I	1.25	0.50	1.50	1.35	--	1.20 (4)	--	--	--	1.00	0.5
Extreme I	0.90	--	1.50	1.00	--	1.20 (4)	--	--	--	1.00	0.5
Extreme I	1.25	0.50	1.50	1.35	--	--	0.90	--	--	1.00	0.5
Extreme I	0.90	--	1.50	1.00	--	--	0.90	--	--	1.00	0.5
Service I	1.00	1.00	1.00	1.00	--	1.00	--	1.40 (5)	1.40 (5)	--	1.00
Service II	1.00	1.30	1.00	1.00	--	1.00	--	1.40 (5)	--	--	--
Service III	1.00	0.80	1.00	1.00	--	1.00	--	1.40 (5)	--	--	1.00
Service IV	1.00	--	1.00	1.00	--	1.00	--	1.40 (5)	1.40 (5)	--	1.00

Notes:

(1) Pedestrian loads will only be significant for platforms and walkways.
(2) Includes buoyancy.
(3) If considering deflections, TU load factor shall be taken as 1.2.
(4) Assigns a higher load factor to account for impact of tides above MHHW.
(5) Use the two year wave and wind condition for service loads.

DC = Dead load of structural components
LL = Vehicular and uniform live load
PLL = Pedestrian and work platform loads
EH = Earth pressure horizontal
EV = Earth pressure vertical
WA = Static water loads
WV = Wave Force
WS = Wind Force
EQ = Seismic Force
TU = Uniform Temperature

7.5 Hydraulic Control Structures

7.5.1 Hydraulic Control Structures Functional Requirements

7.5.1.1. Provide for the safe and controlled flooding and emptying of the pontoon casting basin (see part 11.0 for additional flooding and emptying requirements).

7.5.1.2. Provide screening to exclude fish from entering the basin during flooding

7.5.1.3. Provide for the collection and removal of fish that become trapped in the flooded casting basin after gate closure and returning them to the launch channel (see part 11.0 for additional fish handling and screening requirements).

7.5.2 Hydraulic Control Structures Description

7.5.2.1. Overview

7.5.2.1.1. The hydraulic control structures include the structures and equipment necessary for filling and emptying the casting basin in a controlled manner and consist primarily of an intake structure and a discharge pump shed structure.

The intake structure consists of two 48 inch diameter steel pipes that draw in water from the channel through a removable fish screen box and into the basin via two 48" sluice gates. The pipes and sluice gates are connected via a thimble cast into the concrete wall on the east side of the east jamb. The removable fish screen box is 15 feet wide, 22 feet long and 12 feet tall. Stainless steel profile bar fish screens mounted within the framed panels of the box exclude fish from the intake pipes. The removable intake fish screen box is supported on a 36" diameter steel monopole and will be removed, cleaned, and stored when not in use during basin filling operations. After the intake fish screen box is removed, the open ends of the intake pipes will be sealed with bolt on cover plates. These cover plates will allow the sluice gates to be periodically exercised and maintained without flooding the basin.

The discharge pump shed structure is located on the north side of the east bulkhead wall in the southeast corner of the basin. The steel framed pump shed is 45 feet long and 11.5 feet wide. The roof of the shed is covered by solid steel plates. The north side of the shed is 4 feet tall and contains (6) 4 feet tall x 7.5 foot wide removable stainless steel fish screens. When the basin is being emptied, water is drawn in through these screens by (6) temporary submersible pumps set in a 2 foot deep by 45 foot long sump. The pumps are lowered into location from above through (6) 42 inch diameter HPDE pipe riser casings that are supported from a steel framed access platform at the top of the east bulkhead wall. After the basin is emptied, the (6) removable fish screen panels will be removed, cleaned, and stored outside of the basin.

7.5.2.2. Vertical Layout

7.5.2.2.1. Hydraulic Control Structure Elevations

The following elevations are based on MLLW = 0.0 feet

- Top of pump platforms +18.00 feet
- Casting basin slab -9.00 feet
- Intake pipe invert -8.33 feet
- Bottom of pump sump -11.00 feet

7.5.3 Design Codes and Specifications

7.5.3.1.1. See section 7.1.3

7.5.4 Loads

7.5.4.1. Dead Load, DC

7.5.4.1.1. Self-weight of structure (see section 7.1.5.1 for material densities)

7.5.4.1.2. Submersible pumps weighing 1,200 lbs. each

7.5.4.2. Live Loads, PL

7.5.4.2.1. Roof live load of 20 psf on the discharge pump shed roof

7.5.4.2.2. Platform live load of 100 psf on pump access platform

7.5.4.3. Wind , WS

7.5.4.3.1. Wind loading on the hydraulic control structures is based on a 3-second gust wind speed of 105mph as required by Grays Harbor County Planning and Building Division.

7.5.4.4. Seismic, EQ

7.5.4.4.1. The hydraulic control structures are designed to withstand the 1,000 year earthquake as defined by the site-specific response spectrum provided.

7.5.5 Load Combinations

7.5.5.1.1. See section 7.1.6

7.5.6 Materials

7.5.6.1.1. See section 7.1.7

7.6 Tower Gantry Crane Foundations

7.6.1 Tower Gantry Crane Foundation Functional Requirements

The tower gantry crane foundations are pile supported structures that allow a rail mounted tower gantry crane to pick, trolley, rotate and set objects to aid in the construction of the pontoons on the casting basin slab.

7.6.2 Tower Gantry Crane Foundation Description

7.6.2.1. Overview

The tower crane rails are anchored to a longitudinal concrete beam supported by steel pipe piles. These beams are continuous for the length of the rails and contain transverse cross beams at the piles. Temporary steel beams and a temporary timber deck provided by the Design-Builder allow for a crawler crane or trucks to drive onto a level platform. The beams and the deck will be removed upon completion of the project. The longitudinal, transverse beams and piles will remain after project completion.

7.6.2.2. Horizontal Layout

The spacing of the crane rails is required to be 32.81′ (10m) for the tower crane. The out to out dimension of the longitudinal crane beams is 35.79′, and the total length of the longitudinal beams is 720′ on the east side of the basin, and 755′ on the west side of the basin. Pile bents are 20′ on center. The piles are centered on the longitudinal crane beams.

7.6.2.3. Vertical Layout

7.6.2.3.1. Tower Gantry Crane Foundation Elevations

The following elevations are based on MLLW = 0.0 feet

- Top of gantry crane support structures
  East side +17.00 feet
- Top of gantry crane support structures
  West side +18.00 feet

7.6.3 Design Codes and Specifications

See Section 7.1.3

7.6.4 Loads

7.6.4.1. Dead

7.6.4.1.1. Earth Pressure (EH)

Active horizontal earth pressure due to the soil retained by the upland longitudinal crane beam is applied. The earth pressure is calculated using a γ_s = 120lb/cf, and a k_h value of 0.20.

7.6.4.1.2. Superimposed Dead Load (DL_{SI})

A superimposed dead load has been placed on the tower crane beams and captures the temporary timber crane mats and utilities. The design dead load is 70psf.

7.6.4.1.3. Tower Crane (DL_{TC})

The design tower crane is a Potain MD560A on a rail traveling chassis. The design crane height is 224' with a 263' boom. Vertical and horizontal reactions are provided by the crane manufacturer.

Tower cranes require a spacing of at least 60' when in use.

A 800psf live load is applied at least 20' away from the last wheel of the tower crane trucks, both when picking and in the stowed condition for high wind speeds.

A 200psf deck live load can be applied simultaneously with the tower cranes.

7.6.4.1.4. Crawler Crane (DL_{2250})

The design crawler crane is a Manitowoc 2250 Series 3 with a 190' Boom. It is assumed that the crane does not travel with a pick and that while picking the treads are parallel to the longitudinal crane beams. Crawler Crane Loads were obtained from the Manitowoc Ground Bearing Pressure Estimator Computer program.

7.6.4.2. Seismic (EQ)

Seismic loads are calculated per 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design. A demand model was used to calculate the displacement demand of the structure. The crane ballast near the base of the crane was included in the seismic mass of the structure, as well as vertical load on the piling. It is assumed that the higher mast, boom and counterweights of the crane are out of phase with the structure, and will not contribute to the displacement of the structure. The earthquake resisting system (ERS) of the crane beams consists of first hinging at the pile beam connection, followed by in ground hinging. Although the pile does not meet the ductility requirements for a ductile member in AASHTO, it will be completely filled with concrete to prevent local buckling.

7.6.4.3. Wind (WS)

The design wind speeds for the tower crane represents a nominal 3 second wind gust at 33' above the ground level. Wind loading is determined in accordance with ASCE 7-05, Exposure Category C. The design win speeds are as follows:

7.6.4.3.1. Load Case 1 represents wind speeds up to 45 mph
7.6.4.3.2. Load Case 2 represents wind speeds up to 80 mph
7.6.4.3.3. Load Case 3 represents wind speeds up to 94.5 mph. The wind speed corresponding to the site design wind speed of

105 mph reduced by a factor of 0.9 in accordance with SEI/ASCE 37-02 assuming the cranes are temporary and will be in service for 5 years or less. Note that wind speeds over 80 mph require a positive tie down to the crane beams.

7.6.4.4. Temperature (TU)

- The temperature range shall be 33 degrees up and 33 degrees down.

7.6.5 Load Combinations

AASHTO Load Combinations are used. A hybrid load factor (γ_{cr}) will be used for the crane loads. This load factor accounts for the ratio of dead to live load, wind load, and includes an impact factor of 1.25 for live load during the in-service case. Note that the impact factor is not used for design of piles. Load cases not listed do not control by inspection.

For the extreme event load cases, DL_{TC} represents the dead load of the tower crane acting in the direction of gravity. The horizontal earthquake forces are obtained by using only the seismic mass of the ballast and trucks of the tower crane. A further discussion of the seismic modeling is presented in Section 7.6.4.2.

γ_{cr1} = 1.5 (Composite Load Factor for crawler crane with impact factor)
γ_{cr2} = 1.75 (Composite Load Factor for tower crane including an impact factor)
γ_{cr3} = 1.4 (Composite Load Factor for tower crane without an impact factor)
γ_{cr4} = 1.6 (Composite Load Factor for tower crane for the high wind speed tie-downs)

Strength IA (Out of Service):	$1.25DC + 1.5EH + 1.4\ DL_{2250} + 0.5\ TU$
Strength IB (In Service):	$1.25DC + 1.5EH + \gamma_{cr1}\ DL_{2250} + 0.5\ TU$
Strength 3A (In Service):	$1.25DC + 1.5EH + \gamma_{cr2}\ DL_{TC} + 1.4WS + 0.5\ TU$
Strength 3B (Out of Service 1):	$1.25DC + 1.5EH + \gamma_{cr3}\ DL_{TC} + 1.4WS + 0.5\ TU$
Strength 3C (Out of Service 2):	$1.25DC + 1.5EH + \gamma_{cr4}\ DL_{TC} + 1.6WS + 0.5\ TU$
Extreme 1a:	$1.0DC + 1.0DL_{TC} + 1.0EQ_x + 0.3EQ_y$
Extreme 1b:	$1.0DC + 1.0DL_{TC} + 1.0EQ_y + 0.3EQ_x$
Service 1:	$1.0\ DC + 1.0\ EH + 1.0\ DL_{2250} + 1.0\ TU$
Service 2:	$1.0\ DC + 1.0\ EH + 1.0\ DL_{TC} + 1.0\ WS + 1.0\ TU$

7.6.6 Materials

See Section 7.1.7.

7.7 Pile Supported Crane Work Trestles

TBD

PART 8.0 STOP LOG GATE STRUCTURE

8.1 Design

8.1.1 Stop Log Gate Functional Requirements

8.1.1.1. Provide 110' clear opening

8.1.1.2. The top of gate elevation shall be not lower than the FEMA 50 Year Flood plus 3'. The top of gate elevation was established as = +18.0' MLLW.

8.1.1.3. The time required to remove and install the gate shall be compatible with the planned pontoon construction schedule.

8.1.1.4. The gate corrosion protection system shall have a lifespan of at least 15 years in the marine environment.

8.1.1.5. The gate structure shall be designed and fabricated in accordance with the Mandatory Standards.

8.1.1.6. The gate shall include seals to minimize leakage caused by hydrostatic heads and waves to elevation +18.0' MLLW.

8.1.1.7. Pontoon maneuvering will not occur within the casting basin when the gate is closed during normal operation, which limits the sealing requirements to inward acting hydrostatic heads only.

8.1.1.8. The gate shall be designed to resist catastrophic collapse and sudden flooding of the casting basin. Inelastic deformation and non-repairable damage to the gate may occur during the design seismic event provided that collapse or sudden flooding of the basin does not occur.

8.1.1.9. Non-redundant gate structural elements that are critical to the stability of the gate will be identified as fracture critical members, shall be designated as such on the drawings, and will be designed in accordance with the Mandatory Standards.

8.1.2 Stop Log Gate Description

The stop log gate is a structural steel assembly consisting of a three section, stiffened steel barrier wall supported by three steel trusses. In turn the trusses are laterally supported by truss chord bearing blocks at the jamb interfaces and by vertical supports at the casting basin slab. A perimeter seal along bottom and two vertical sides of the barrier wall, and intermediate seals, one each separating the three barrier wall sections, minimize leakage into the casting basin. The weight of each sub-assembly is limited to 50 tons to facilitate installation, removal and storage. Screw jacks are used to hold the gate against the jambs after the perimeter seal is compressed to minimize leakage. Hydraulic jacks located between the basin side face of each jamb and the bottom chord of the gate lower truss are used to preload the gate perimeter seal significantly to prevent leakage and allow basin dewatering to compress the seal. Additional blocking is used to provide longitudinal positioning of the gate with respect to the centerline of the casting basin opening while not restricting gate thermal lengthening and shortening.

The barrier wall at the lower truss assembly is allowed to rotate relative to the upper barrier wall segments by a spring loaded connection at the lower chord of the lower truss and by a rubber bearing at the upper chord of the lowest truss. Belleville springs are used to control the magnitude of the reaction forces into the lower truss.

Each barrier wall segment/supporting truss transfers lateral and longitudinal shear forces and axial vertical forces to adjacent trusses. Lateral and longitudinal shear forces are transferred between trusses by shear transfer fittings. Axial tension forces are transferred by thread bars adjacent to the front and back truss chords, and axial compression forces are transferred at truss vertical member interfaces.

The perimeter seal and intermediate seals consist of a compressible molded natural rubber seal which encapsulates a steel mounting plate. The fastening system provides the capability to remove and replace portions of the seal if local damage occurs. The perimeter seal is a compliant element that is compressed to prevent leakage, allowing lateral forces on the barrier wall to be transferred to the trusses and then to a series of bearing blocks and into jambs. Over compression of the perimeter seal is prevented by seal stops located on the barrier wall perimeter. The intermediate seals are located within the horizontal joints in the exterior barrier wall. These compliant seals provide sealing when compressed and are prevented from over compression by seal stops affixed to the inside face of the barrier wall.

The gate is vertically supported at four locations on the casting basin slab. The locations are at the intersection of Gridlines A3, B3, A7 and B7. Each support location consists of a stainless steel plate anchored to the casting basin slab. UHMW pads attached to the gate bear on the stainless steel plate to minimize the friction coefficient between the gate and the slab. The friction coefficient at the interface is expected to vary between 0.10 and 0.20.

8.1.3 Tidal Elevations

The following tides are taken from NOAA Station #9441187, Aberdeen, WA and are measured using a datum of MLLW = 0.0 feet

➢ FEMA 50 Year Flood	+14.90 feet
➢ Highest Observed Tide (HOT)	+13.86 feet
➢ Mean Higher High Water (MHHW)	+10.11 feet
➢ Mean High Water (MHW)	+9.41 feet
➢ Mean Tide Level (MTL)	+5.60 feet
➢ Mean Sea Level (MSL)	+5.44 feet
➢ Mean Low Water (MLW)	+1.47 feet
➢ Mean Lower Low Water (MLLW)	+0.00 feet
➢ Lowest Observed Tide (LOT)	-3.35 feet

8.1.4 Structure Elevations

The following elevations are based on MLLW = 0.0 feet

➢ Top of Gate (At Barrier Wall)	+18.00 feet
➢ Top of Gate Perimeter Seal	+ 18.00 feet
➢ Bottom Gate Vertical Support	- 9.00 feet
➢ Top of Casting Basin Slab	- 9.00 feet
➢ Centerline of Gate Lower Perimeter Seal	- 9.50 feet
➢ Top of Sill at Back Chord of Gate	- 9.00 feet
➢ Top of Sill at Front Chord of gate	- 9.00 feet
➢ Bottom of Launch Channel	- 13.00 feet
➢ Minimum Tide Elevation for Pontoon Launch	+ 9.00 feet

- Design Tide Elevation for Gate Assembly +10.11 feet

8.1.5 Design Codes and Specifications

16. WSDOT Standard Specifications and Amendments, (M41-10), Latest Edition, Washington State Department of Transportation.
17. WSDOT Bridge Design Manual LRFD (M23-50), Latest Edition, Washington State Department of Transportation.
18. WSDOT Geotechnical Design Manual (GDM), Revised Chapter 6 and Chapter 8, August 14, 2009, Washington State Department of Transportation.
19. WSDOT Geotechnical Design Manual (M46-03), Latest Edition, Washington State Department of Transportation.
20. WSDOT Design Manual (M22-01), Latest Edition, Washington State Department of Transportation.
21. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, Fourth Edition (2007), with 2009 Interim Revisions, American Association of State Highway and Transportation Officials.
22. 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design, American Association of State Highway and Transportation Officials.
23. IBC International Building Code, (2006), International Building Code Council, Inc., as modified by the Washington State Building Code Council.
24. Minimum Design Loads for Buildings and Other Structures, ASCE / SEI 7-05, American Society of Civil Engineers.
25. AWS D1.1 Structural Welding Code - Steel (2008), American Welding Society.
26. AWS D1.5 Bridge Welding Code (2008), American Welding Society.
27. LRFD Guide Specifications for the Design of Pedestrian Bridges NCHRP 20-07 Task 244.

8.1.6 References

1. Military Handbook M1L – HDBK -149A "Rubber and Rubber Like Materials", U.S. Army Materials and Mechanics Research Center, June 1965.
2. SR 520 Pontoon Construction Design-Build Project, Chapter 2, Technical Requirements, Request for Proposal, August 24, 2009.
3. SR 520 Pontoon Casting Facility Coastal Engineering Report, Coast and Harbor Engineers, January 14, 2010
4. SR 520 Pontoon Casting Facility Geotechnical Report, Shannon and Wilson, January 14, 2011

5. Design of Marine Facilities for the Berthing Mooring, and Repair of Vessels Second Edition, Gaythwaite, John W. ASCE Press, American Society of Civil Engineers.

8.1.7 Operational Loading

8.1.7.1. Performance Criteria

8.1.7.2. Load Cases

8.1.7.2.1. Steel Frame

The gate is subject to vertical and lateral loads in the operational condition. The vertical loads include dead load of the gate, buoyancy created by the perimeter seal and closed tubular members, a live load on the catwalk located at the top truss of the gate and forces induced by vertical seismic accelerations. The horizontal loads include hydrostatic loads, wind loading, wave loading, thermal movement/restraint, and transverse and longitudinal forces induced by seismic accelerations.

The critical design hydrostatic load is the water pressure based on the FEMA 50 year flood, and is defined in coastal engineering report. Other hydrostatic loads are also considered in the design. Wind loading is based on a (3) second wind gust with a 50 year return period, and a base wind speed of 105 mph as defined in the coastal engineering report. The wave loading on the gate is based on waves from the south, and a 50 year return period. Wave heights and periods are per the coastal engineering report

The catwalk design live load is 85 psf.

Because the gate is virtually unrestrained in the east-west direction, the gate has been designed to slide longitudinally until it contacts either of the jambs. The gate contact points consist of UHMW bearings located on the grid 1.2 and 2.8 tubular members located on T3U, T3L, T2U, T2L and T1U. When the gate is centered in the 110' opening between the jambs, the gate will slide at its vertical support locations approximately 2" until it contacts the jambs. The contact point reactions at the jambs are carried through the three part truss system.

The gate is designed for a thermal gradient across the face. The gradient across the wall is 40° F with the channel side at 50° F and the basin side at 90° F. The screw jacks holding the gate against the jambs utilize spherical washers which decouple jack bending as the gate expands and contracts longitudinally. Therefore, the gate itself is virtually unrestrained in the longitudinal direction.

The trusses span horizontally between the jambs. The primary load carrying members in the gate trusses are closed sections. All closed sections will be capped and seal welded to prevent corrosion on interior surfaces. The barrier wall spans vertically between each truss top and bottom chord. It is a stiffened flat steel plate. The type and size of the stiffeners varies with load demand on the wall. All open sections will be provided with drain holes.

Load cases are per AASHTO LRFD Bridge Design Specifications and are provided in the following. Acronyms for load sources conform to AASHTO definitions.

Strength I

STR1A: $1.25\ DC + 1.75\ PL + 1.0\ (WA + Buoyancy)_{flood} + 1.0\ TG$
STR1B: $0.90\ DC + 1.75\ PL + 1.0\ (WA + Buoyancy)_{flood} + 1.0\ TG$

Strength II

By inspection, Strength II load combinations are less critical than Strength I combinations.

Strength III

STR3A: $1.25\ DC + 1.0\ (WA + Buoyancy)_{flood} + 1.40\ WV + 1.40\ WS + 1.0\ TG$
STR3B: $0.90\ DC + 1.0\ (WA + Buoyancy)_{flood} + 1.40\ WV + 1.40\ WS + 1.0\ TG$

STR3C: $1.25\ DC + 1.2\ (WA + Buoyancy)_{mhhw} + 1.40\ WV + 1.40\ WS + 1.0\ TG$

STR3D: $0.90\ DC + 1.2\ (WA + Buoyancy)_{mhhw} + 1.40\ WV + 1.40\ WS + 1.0\ TG$

Strength IV

STR4A: $1.50\ DC + 1.0\ (WA_{flood}) + 1.0\ TG$

Strength V:

By inspection, Strength V load combinations are less critical than Strength III combinations.

The following service limit state combinations were used for the design of the Belleville springs and other moving parts of the gate.

Service IA: $1.0\ DC + 1.0\ PL + 1.0\ (WA + Buoyancy)_{flood} + 0.3\ WV + 0.3\ WS + 1.0\ TG$

Service II: $1.0\ DC + 1.3\ PL + 1.0\ (WA + Buoyancy)_{mhhw} + 1.0\ TG$

Service IV: $1.0\ DC + 1.0\ (WA + Buoyancy)_{mhhw} + 0.7\ WV + 0.7\ WS + 1.0\ TG$

8.1.7.2.2. Perimeter Seal

The perimeter seal is comprised of a molded, 30-durometer, natural rubber seal vulcanized around a steel attachment plate. The seal is compressed when the gate is moved into position against the jambs using a combination of hydraulic rams and screw jacks. The hydraulic rams and screw jacks provide sufficient compression to allow the seal to remain stable (not blow out laterally or vertically) when compressed against bearing plates embedded in the sill face and the jambs. Seal stops located adjacent to the perimeter seal along the horizontal sill and the vertical jamb faces will bear against the sill and jambs when the seal is deflected to its specified compression. The designed specified compression includes the required compression to minimize leakage and an allowance for gate and sill/jamb fabrication tolerances. Once the gate bearing blocks are seated and the gate is locked against the jambs, the seal will maintain its specified compression and all additional lateral forces will be transferred through the bearing blocks to the sill and jambs. The rigid bearing blocks also prevent inadvertent over-compression of the perimeter seal during installation of the gate. The design water immersion hydrostatic head used for seal design is 24.5 feet, accounting for a sustained water level at elevation = +15.0 ft. and the seal located near the bottom of the barrier wall at elevation – 9.5 ft. Temporary wave encounters at elevations above the FEMA 50 Year Flood stage are not used in the seal design, although the seal is expected to provide adequate sealing during this condition. The rationale for this approach is that increases in hydrostatic pressure at the bottom of the barrier wall due to wave encounter are minimal. The perimeter seal is considered to be a maintenance item that may require repair or replacement, and will be designed for removal and replacement.

8.1.7.2.3. Intermediate Seals

The two intermediate seals are similar in shape and appearance to the perimeter seal, but have different load /deflection properties. These seals are also made from molded 30-durometer natural rubber vulcanized around a steel attachment plate. Both seals are designed to reach their specified design compression when loaded only by the vertical dead load from the gate section(s) above. The specified compression includes the required compression to minimize leakage plus and allowance for gate fabrication tolerances. The design vertical dead loads include an allowance for the buoyancy of the gate sections during gate assembly and installation. Over compression of these seals is prevented by seal stops affixed to the inside face of the barrier wall. Vertical threadbars are used to transfer loads throughout the throughout the three truss assemblies. The threadbars extend from top chord of truss T3 to the top chord of truss T1, and therefore span across both horizontal joints (and intermediate seals) in the barrier walls. The threadbars provide a positive clamping force to maintain intermediate seal compression. The design hydrostatic head height for the lower intermediate seal is 19.8 feet. The design hydrostatic head height for the upper intermediate seal is 11.2 feet. The intermediate seals are considered to be maintenance items that may require repair or replacement, and will be designed for easy removal and replacement.

8.1.7.2.4. Lifting Points and Storage

Lifting and storage support points are designed into each gate truss section. The lifting points will be consistent with the Contractor's plan for lifting. Weight and center of gravity data for each gate truss section are provided on the plans. Also, because each gate truss section will be either completely or partially submerged during installation, buoyancy forces will cause a reduction in the lift load and a shift in the center of gravity of each truss section upon immersion. These data are also provided on the plans.

8.1.8 Seismic Loading

8.1.8.1. Seismic Demand

The gate is designed to withstand the 1000 year earthquake defined by the project site-specific response spectrum. The minimum response shall not be less than 2/3 of the Code-Based AASHTO 1000 year event. The gate is designed to resist catastrophic collapse and sudden flooding of the casting basin.

8.1.8.2. Load Cases

8.1.8.2.1. Steel frame

The gate is subject to seismic forces in both the horizontal and vertical directions. The design seismic event is based on the 1,000 year earthquake and the forces are based on the site specific response spectrum shown in the Geotechnical Report. The seismic load is applied concurrently with a water pressure equivalent to a water elevation of MHHW. A seismic event in the 'X' direction is along the length of the gate. A seismic event in the positive 'Y' direction acts toward and away from the basin, and a seismic event in the positive 'Z' direction acts in the direction of gravity. Load cases are per AASHTO LRFD Bridge Design Specifications and are as follows:

Extreme Event I

EXT1A: $1.25\ DL + 0.50\ PL + 1.0\ (WA + Buoyancy)_{MHHW} + 1.0\ EQX + 0.3EQY + 0.2EQZ$

EXT1B: 0.90 DL + 1.0 (WA + Buoyancy) $_{MHHW}$ *+ 1.0 EQX + 0.3EQY - 0.2EQZ*

EXT1C: 1.25 DC + 0.50 PL + 1.0 (WA + Buoyancy) $_{MHHW}$ *+ 1.0 EQY + 0.3EQX + 0.2EQZ*

EXT1D: 0.9 DC + 1. 0 (WA + Buoyancy) $_{lot}$ *+ 1.0 EQY + 0.3EQX - 0.2EQZ*

EXT1E: 0.90 DC + 1.0 (WA + Buoyancy) $_{lot}$ *- 1.0 EQY + 0.3EQX + 0.2EQZ*

8.1.9 Materials and Corrosion Protection

8.1.9.1. Structural Carbon Steel

Unless noted otherwise:

- MC, C, and L shapes shall be AASHTO M270 Grade 36 or AASHTO M270 Grade 50 where required
- Plate shall be AASHTO M270 Grade 36 or AASHTO M270 Grade 50 where required
- Rectangular HSS shall be ASTM A500 Grade B, Fy = 46ksi
- Round HSS shall be ASTM A500 Grade B, Fy = 42ksi
- Welding shall be done using E70XX Electrodes

8.1.9.2. Stainless Steel

- Stainless steel plate shall be ASTM A240, Type 316L

8.1.9.3. Ultra High Molecular Weight Polyethylene (UHMW)

- ASTM D4020

8.1.9.4. Corrosion Protection – Carbon Steel Members

- Structural steel elements, connections, fittings, etc. shall have corrosion protection for a design life of 15 years in a marine environment. A marine grade multi-purpose epoxy system will be used for carbon steel members. Wearing surfaces, for example at gate support locations on the basin slab and truss to jamb interfaces, will be UHMW. Carbon steel threaded rods, nuts and washers shall be galvanized. Isolation washers and bushings shall be used at carbon to stainless steel interfaces.

8.1.9.5. Natural Rubber

The seal assemblies consist of a molded rubber seal, an encased steel plate, and nuts and washers that connect the seal to threaded studs.

All molded rubber for the seals shall be from the same lot of rubber. Test samples will be required for testing in accordance to the Special Provisions. The rubber seal material shall be natural rubber containing carbon black, zinc oxide, accelerators, antioxidants, vulcanizing agents and plasticizers. The physical characteristics shall be in conformance with the Special Provisions. The encased base plate shall conform to ASTM A240, Type 316L. Studs shall conform to ASTM F593, Type 316. Washers shall conform to ANSI B.18.22.1, Type 316.

Rubber bearing pads shall be a compound of natural rubber containing carbon black, zinc oxide, accelerators, antioxidants, vulcanizing agents and plasticizers. The physical characteristics shall be in conformance with the Special Provisions.

8.1.9.6. Springs

- The moveable spring assemblies located at the T1L truss connection include UNS 45000 stainless steel Belleville washer assemblies, carbon steel thread bars, nuts, washers, isolation washers and bushings.

8.1.9.7. Spherical Washers

- Spherical washers used in the screw jack assemblies shall have a minimum total rotation angle of ± 2°.

8.1.10 Gate Fit Up

8.1.10.1. General

The sill and jambs will be constructed in the dry behind a cutoff wall. The gate also will be fitted to the as-constructed sill and jambs in the dry. The majority of the sill / jamb construction can be completed using standard concrete construction tolerances. The majority of the gate fabrication will be completed offsite, and can be completed using standard steel fabrication tolerances. However, the discrete gate to sill and jamb interface locations will require special tolerance consideration, and these locations have been designed to provide adjustment capability during the gate fit-up sequence. Interface adjustment capability will be required at the following locations:

- The gate vertical support locations at Grids A and B, and 3 and 7 on the casting basin slab.
- The discrete gate truss bearing locations on the jambs.
- The continuous gate perimeter seal bearings on the sill and jamb faces.

Successful fit-up between the gate and the sill /jambs are intended to accomplish three goals:

- Provide adequate sealing in the lower one-third of the gate height to allow for initial de-watering of the casting basin during gate assembly.
- Provide the required load paths between the gate and the sill / jamb structures.
- Minimizing leakage at the perimeter seal during casting basin pontoon construction.

The recommended gate to sill and jamb fit-up sequence is summarized on the Gate Drawings.

8.1.11 Operation

8.1.11.1. Installation

- The gate will be installed in (3) sections, the bottom section (T1), middle section (T2), and top section (T3).
- The lower gate section (T1) will first be installed during a falling tide. T1 will be lowered down, with the front and back chords astride of the jambs. The clear distance between the inside flanges of the truss chords is established to allow for the thickness of the seals (uncompressed), the bearing block thickness, truss rotation on immersion, and adequate space for installation of a jack system for positioning of the gate prior to installing the blocking to provide lateral and longitudinal support. The top of T1 and T2 will have tapered alignment shear transfer fittings that will engage T2 and T3 as they are lowered around the jambs. When the tapered fittings are engaged and T2 is lowered down on T1, the deadweight of T2 will compress the horizontal seal between T2 and T1. T3 is then lowered into T2 in a similar fashion, compressing the horizontal seal between T2 and T3.

- The complete gate must be assembled vertically prior to moving the gate to contact the sill and jambs. The gate will be installed away from the sill, and using the hydraulic jacks, pulled against the sill and jambs to compress the perimeter seal and achieve bearing at the bearing blocks and water sealing over the lower portion of the gate weight. Hydraulic jacks located between the inside of the jambs and grid B T1L truss chord provide this capability. At this point the basin dewatering can begin.
- Basin dewatering will begin as the tide begins to rise. The rising tide will compress the perimeter seal against the jamb faces to the specified amount. The screw jacks are used to complete compression of the perimeter seal and to fully seat the truss bearing blocks. Blocking can be added as required to maintain the gate position as the hydraulic jacks are unloaded.

8.1.11.2. Removal

- To remove the gate, the water heights on either side of the gate must be equal. All blocking and screw jacks used to maintain gate location in service shall be removed. At this point, the gate can be pushed away from the sill and jamb. The gate segments can then be lifted and removed. Hydraulic jacks located behind the Grid A – T1L truss chord will react against the outside face of the jambs to provide this movement.

8.1.11.3. Storage

- When not in use, the gate is to be stored on site, blocked and stored in the upright position. During storage periods the seals and UHMW pads can be inspected and repaired or replaced as needed.

8.1.11.4. Transportation / Delivery

- The gate subassemblies will be transported to the project site by surface transportation. Each of the three subassemblies will include the trusses, barrier wall, seals and UHMW pads that interface with the embedded plates located on the casting basin sill and jambs.

PART 9.0 BASIS OF DESIGN – PONTOONS

9.1 Design Overview

Pontoons to be designed by WSDOT

PART 10.0 BASIS OF DESIGN – LAUNCH CHANNEL

10.1 Launch Channel Dolphins

10.1.1 Launch Channel Dolphins Functional Requirements

The launch channel dolphins shall assist in the launching of the pontoons, berthing and mooring of the pontoons, and guidance and turning assistance for the pontoons.

10.1.2 Channel Description

10.1.2.1. Horizontal Layout

The launch channel extends SE from the gate opening out approximately 600 feet into the Chehalis River. At the southern end, the channel flares west to provide an area to rotate pontoons to be roughly parallel with river flow. The flat portion of the channel is approximately 135 feet wide and is dredged to a -13' MLLW elevation. The channel slopes up to match grade on the east and west sides with a protected cut slope. Guide dolphin piles are provided at approximately 20' on center along both sides of the float out channel. A continuous steel wide flange will be attached to the top of all the guide piles and will act as a rubstrip to aid in towing out the pontoons. One large turning dolphin will be provided on each side of the channel to provide a rest point for the pontoons, one for flood tide loadout and the other for ebb tide loadout. A battered pile turning dolphin was designed on the flood tide/ downstream side of the launch channel to reduce deflection and allow the full height of the pontoon wall to be utilized to distribute the forces across the full pontoon depth. On the ebb tide side of the launch channel a mono pile was designed because the forces are a lot less and deflection isn't as much of an issue.

10.1.2.2. Vertical Layout

- Tidal Elevations

  The following tides are taken from NOAA Station #9441187, Aberdeen, WA and are measured using a datum of MLLW = 0.0'

  - Highest Observed Tide (HOT) +13.86 feet
  - Mean Higher High Water (MHHW) +10.11 feet
  - Mean High Water (MHW) +9.41 feet
  - Mean Tide Level (MTL) +5.44 feet
  - Mean Low Water (MLW) +1.47 feet
  - Mean Lower Low Water (MLLW) +0.00 feet
  - Lowest Observed Tide -3.35 feet

- Launch Channel Elevations

  The following elevations are based on MLLW = 0.0 feet

  - The channel must be a minimum of 3'-3" below the bottom of the gate structure to prevent fouling of the gate seals and rest logs.
  - The channel must be a minimum of 1' below the elevation of the bottom water intake screen.

10.1.3 Design Codes and Specifications

28. WSDOT Bridge Design Manual LRFD (M23-50), Latest Edition, Washington State Department of Transportation.
29. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, Fourth Edition (2007), with 2009 Interim Revisions, American Association of State Highway and Transportation Officials.
30. 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design, American Association of State Highway and Transportation Officials.
31. AISC Manual of Steel Construction, 13th Edition, (2005), American Institute of Steel Construction.
32. IBC International Building Code, (2006), International Building Code Council, Inc., as modified by the Washington State Building Code Council.
33. Minimum Design Loads for Buildings and Other Structures, ASCE / SEI 7-05, American Society of Civil Engineers.

34. ACI Building Code Requirements for Structural Concrete with Commentary, ACI 318-08, American Concrete Institute.
35. Guide for the Design and Construction of Fixed Offshore Concrete Structures, ACI 357R-84 (Reapproved 1997), American Concrete Institute.
36. U.S. Department of Defense, UFC 4-213-10, "Design: Graving Docks".
37. SR 520 Pontoon Construction Design-Build Project, Chapter 2, Technical Requirements, Request for Proposal, August 24, 2009.
38. SR 520 Pontoon Casting Facility Coastal Engineering Report, Coast and Harbor Engineers, January 14, 2010
39. SR 520 Pontoon Casting Facility Geotechnical Report, Shannon and Wilson, January 14, 2011
40. Design of Marine Facilities for the Berthing Mooring, and Repair of Vessels Second Edition, Gaythwaite, John W. ASCE Press, American Society of Civil Engineers.
41. WSDOT Standard Specifications and Amendments, (M41-10), 2008, Washington State Department of Transportation.

10.1.4 Loads

10.1.4.1. Material Densities

- Reinforced Concrete 160 pcf
- Seawater 64 pcf
- Steel (including stainless) 490 pcf

10.1.4.2. Wind Loading: Easterly 40.0 mph, 3 second wind gust using AASHTO Equation 3.8.1.2.1-1 and AASHTO Wind Load (WS) load factor. This wind load corresponds to a 2 year return period per the coastal engineering report.

10.1.4.3. Wave Loading: H_{m0} = 1.0' and T_P = 1.9 seconds using the AASHTO Wind Load (WS) load factor. Wave heights and periods are per the coastal engineering report.

10.1.4.4. Current Loading: Flood tide design current = 2.0 knots. Ebb tide design current = 3.0 knots per Costal Harbor Engineer Report.

10.1.4.5. Berthing Loading: 0.31 ft/sec sheltered berthing velocity as defined by UFC 4-152-01. Total berthing energy = 31 kip-ft.

10.1.4.6. Channel dolphins that are connected by the waler are to resist the lateral movement of the pontoon from the wind and current forces while in the channel. Channel dolphins are not designed for collision forces but are designed for a controlled sheltered berthing.

10.1.4.7. Turning dolphins at the end of the channel are used to resist the lateral movement of the pontoon from the wind and current forces while the pontoon is moving out of the channel. They are also designed to allow the pontoon to rotate while resting against them. Turning dolphins are not designed for collision forces.

10.1.4.8. Tug mooring piles are considered temporary and therefore have no specific loading criteria. There purpose is to be a marker for the tugs and used to temporary rest the tugs against.

10.1.5 Load Combinations

The loads listed in 10.1.4.1 – 10.1.4.5 are assumed to occur in combination. Load factors and combination considered in the design are based on Table 3.4.1-1 from the AASHTO LRFD Bridge Design Specifications, unless noted otherwise.

- AASHTO Strength I
- AASHTO Strength II
- AASHTO Strength III
- AASHTO Strength IV
- AASHTO Strength V
- AASHTO Extreme Event I
- AASHTO Extreme Event II
- AASHTO Service I

Load case assumptions are as followed:

Strength I

STR1A: $1.25\ DC + 1.0\ (WA_{current})$
STR1B: $0.90\ DC + 1.0\ (WA_{current})$

Strength II

By inspection, Strength II load combinations are less critical than Strength I combinations.

Strength III

STR3A: $1.25\ DC + 1.0\ (WA_{current}) + 1.40\ WS$
STR3B: $0.90\ DC + 1.0\ (WA_{current}) + 1.40\ WS$

Strength IV
By inspection the wind and current loading in Strength III will control.

Strength V:
By inspection, Strength V load combinations are less critical than Strength III combinations.

Extreme Event I & II
By inspection, Earthquake load combinations are less critical than Strength III combinations.

Service I

Service1: $1.0\ DC + 1.0\ (WA_{berthing})$

10.1.6 Materials

10.1.6.1. Structural Carbon Steel

Unless noted otherwise:

- W and HP shapes shall be AASHTO M 270 Grade 50
- M, MC, C, S and L shapes shall be AASHTO M270 Grade 36 or AASHTO M270 Grade 50 where required
- Plate shall be AASHTO M260 Grade 36 or AASHTO M270 Grade 50 where required
- Rectangular HSS shall be ASTM A500 Grade B, Fy = 46ksi
- Round HSS shall be ASTM A500 Grade B, Fy = 42ksi
- Pipe shall be ASTM A53 Grade B, Fy = 35ksi
- Fasteners shall be High Strength ASTM A325,galvanized
- Friction type connections: Class B coating on faying surfaces

10.1.6.2. Stainless Steel

- Stainless steel plate shall be ASTM A240, Type 316L
- Stainless steel shims shall be ASTM A240, Type 316

10.1.6.3. Piling

- Steel pipe pile, ASTM A252, Gr 2 45 ksi or 55 ksi,
  ASTM A252, Gr 3 Conforming to Table 3.1 in the AWS D1.1/D1.1M, Latest Edition

10.1.6.4. Ultra High Molecular Weight Polyethylene (UHMW)

- ASTM D4020

10.1.6.5. Corrosion Protection

- Structural steel elements, connections, fittings, etc. shall have corrosion protection for a design life of 15 years in a marine environment.
- Steel piling need not have corrosion protection below an elevation of 25' below the lowest adjacent ground level or an assumed section loss of 0.005" per year.
- Structures and equipment in or adjacent to the launch channel shall be of Type 316L stainless steel or equally corrosion resistant material, except for guide piles, and piling, structural shapes and plates used to construct guide and turning dolphins

10.1.7 Operational features

10.1.7.1. The pontoons will be pulled from the basin using a large tug and guided by smaller tugs from behind. The pontoons will be allowed to rest against the channel dolphins due to the action of the downstream current. The tugs will pull the pontoons along the channel dolphins until the pontoon is supported by only the tug and the last mooring dolphin. At that point a second large tug will assist in moving the pontoon out of the channel and rotating it parallel to the current.

10.1.7.2. The concrete pontoon walls will not be overloaded per the RFP during tug operations as the pontoon is pulled out of the channel.

10.1.7.3. Two graduated tide staffs will be placed on channel dolphins to monitor the tide during float out operations to verify that there is a minimum of 1' clearance between pontoon keel slabs and the channel floor.

10.1.7.4. An electronic tide gauge will be installed on one of the channel dolphins that will collect and record data in at least 6 minute increments.

10.1.7.5. An electronic device for measuring current will be installed at the site to verify current design assumptions.

10.1.7.6. Regular maintenance dredging will be required per the Project Sedimentation Report. Basin operator will evaluate the need for maintenance dredging prior to each load out and perform dredging if required. This will be addressed further in the Operations and Maintenance Manual.

10.2 Launch Channel Civil Design

10.2.1 Launch Channel Civil Design Functional Requirements

From RFP Section 2.13.3.2.4

A launch channel shall be provided that connects the adjacent navigable waterway to the casting basin at its gate location(s).

The channel width at the bottom of its side slopes shall be at least as wide as the associated clear opening of the gate system.

Armoring shall be provided to prevent scour from undermining adjacent side slopes and PCF structures by action not limited to waves, currents, tides, ship wake, ship surge, propeller wash, and the Design-Builder's activities.

The design elevation of the channel bottom, including scour protection, shall be no higher than 3 feet - 3 inches below the top of the gate sill to prevent fouling of the gate system and no higher than 1 foot - 0 inches below the bottom of the water intake for the HCS.

The launch channel shall be configured and maintained so that pontoons with 17 feet - 0 inches of draft can be floated through the launch channel with a minimum of 1 feet -0 inches of clearance between the pontoon keel and the channel bottom.

Sedimentation is expected to occur within the channel. The channel shall be configured to allow for regular maintenance dredging to maintain the required functionality of the channel, including, but not limited to, maintaining the design channel bottom elevation.

At least two graduated tide staffs shall be installed within or adjacent to the launch channel. The tide staffs shall be installed and marked so that the water levels may be read from the staff relative to MLLW for the full range of water levels. An electronic tide gauge shall also be installed and calibrated to automatically report water surface elevation within the launch channel at an interval not to exceed six-minutes. The tide gauge readings shall be transmitted *to a personal computer or handheld electronic device for in real-time monitoring of water levels in the launch channel. Collected data shall be stored by the Design-Builder for submittal to WSDOT.*

From RFP Section 2.13.4.2.4

Armoring shall be designed to remain intact and not migrate outside of the original placement area.

From RFP Section 2.13.5.2.4

Design-Builder shall conduct construction operations in accordance with the general regulations of the U.S. Department of the Army and the U.S. Coast Guard, as applicable.

Prior to construction of the launch channel, the Design-Builder shall perform a hazard survey of the area within the launch channel footprint using a gradiometer, magnetometer, sonar or other applicable method as approved by the Engineer of Record, and submit the results to WSDOT.

The Design-Builder shall perform a pre-dredging and post-dredging hydrographic survey of the launch channel in conjunction with each dredging event.

10.2.2 Channel Description

10.2.2.1. Horizontal Layout

The launch channel extends SE from the gate opening into Grays Harbor. The channel is approximately 600 feet long. At the southern end, the channel flares west to provide an area to rotate pontoons to be roughly parallel with river flow. The flat portion of the channel is approximately 150 feet wide and is dredged to a -13' MLLW elevation. The channel slopes up to match grade on the east and west sides with a protected cut slope per the Geotechnical Engineering Report and the Coastal Engineering Report. A pile supported whaler beam will be provided along the west side of the channel to aid with pontoon float out. Timber piles will be provided at the toe of the slope along the east side of the channel.

10.2.2.2. Vertical Layout

- Tidal Elevations

  The following tides are taken from NOAA Station #9441187, Aberdeen, WA and are measured using a datum of MLLW = 0.0'

  - Highest Observed Tide (HOT) +13.86 feet
  - Mean Higher High Water (MHHW) +10.11 feet
  - Mean High Water (MHW) +9.41 feet
  - Mean Tide Level (MTL) +5.44 feet
  - Mean Low Water (MLW) +1.47 feet
  - Mean Lower Low Water (MLLW) +0.00 feet
  - Lowest Observed Tide -3.35 feet

- Launch Channel Elevations

  The channel will be dredged to an elevation of -13'. This elevation provides the minimum clearances of 1' between the bottom of the intake fish screen and the channel and 3'3" between the channel and the bottom of the gate structure as required in the RFP document.

10.2.3 Channel Slope

The channel sides will be sloped per the Geotechnical Engineering Report. The maximum allowable armored slope is 3:1. The maximum allowable unarmored slope is 5:1. See Geotechnical Engineering Report for more information.

10.2.4 Slope Armor

Slope armor section per the Coastal Engineering Report and Geotechnical Engineering Report. Slope armor will consist of a 3 ft thick heavy rip rap section underlain by a 1 ft thick bedding layer and 2 ft thick light loose rip rap layer. This section is for slope stability. Section required from Coastal Engineering Report is smaller. Rip Rap size: D_{50} = 1.5 ft, W_{50} = 375 Lb for angular rock with density = 170 PCF. See Geotechnical Engineering and Coastal Engineering Reports for more information.

10.2.5 Material Handling and Water Quality Protection

1. Dredge operation requirements are outlined in the RFP document technical requirements section 2.8.4.3.10. A Dredge Material Management and Disposal Plan is required 90 days prior to dredging.
2. Appendix C1 of the RFP document indicates additional notification and dredging requirements.
3. WAC 220-110-120. Dredge area shall be protected and isolated from fish.
4. WAC 220-110-050. Bank Protection Plan required.

10.2.6 Sedimentation

The sedimentation rates vary depending on the location in the launch channel and also with time. It is expected that the most severe sedimentation will occur nearest the navigation channel and immediately after dredging activities. Lower rates are expected nearer the basin gate. Rates are expected to range from 0.72 ft of sediment per month near the navigation channel to 0.12 ft per month near the basin gate. See Coastal Engineering Report for more information.

10.2.7 Survey

Pre and post dredge surveys are required for each dredge event. Dredge cut volumes and disposal location for dredge spoils are required for each dredge event.

10.2.8 In-Water Work Schedule

In-water work (below OHW/MHHW) is only allowed between June 15th and February 28th annually.

10.2.9 Design Codes and Specifications

1. SR 520 Pontoon Construction Design-Build Project, Chapter 2, Technical Requirements, Request for Proposal, August 24, 2009.

PART 11.0 BASIS OF DESIGN – MECHANICAL

11.1 Design Overview

11.1.1 Component Requirements for Emptying Basin

- 11.1.1.1. 6-pumps capable of pumping at 4000gpm against 40 ft of head
- 11.1.1.2. Pump well
- 11.1.1.3. Fish Box – screened and aerated
- 11.1.1.4. Discharge piping to launch channel

11.1.2 Component Requirements for Flooding Basin

- 11.1.2.1. 2-48" Flood pipes
- 11.1.2.2. Sluice gates
- 11.1.2.3. Intake fish screen
- 11.1.2.4. Mono-Pile
- 11.1.2.6. Flow monitoring equipment

11.1.3 Flooding Procedures

- 11.1.3.1. Clean basin
- 11.1.3.2. Place intake fish screen box on flood pipes
- 11.1.3.3. Open sluice gates - Regulate flow by controlling sluice gate openings. Velocity at fish screen will be maintained at 0.4 fps or less. O&M manual will provide more detail
- 11.1.3.4. Allow basin water surface to equilibrate with tidal water surface in channel
- 11.1.3.5. Remove basin gate
- 11.1.3.6. Basin continues to fill as tide rises
- 11.1.3.7. Sluice gates may be closed and intake fish screen removed
- 11.1.3.8. Float out

11.1.4 Emptying Procedures

- 11.1.4.1. Complete float out
- 11.1.4.2. Allow basin to drain through basin gate as the tide falls
- 11.1.4.3. Begin placing basin gate sections at low low water
- 11.1.4.4. 4000 gpm pumps may be started once gate is in place
- 11.1.4.5. Pump basin water surface down to the prescribed fish handling level

11.1.4.5.1 Pumps may need to be shut off as pump screen is exposed to avoid exceeding the 0.4 fps velocity across the fish screen. This is only a requirement prior to the removal of fish.

11.1.4.6. Fish handling and removal

11.1.4.7. Pump out remaining water in basin with 4000 gpm pumps

More detailed operational procedures will be provided in the Operations and Maintenance Manual (O&M).

11.2 Design Requirements

11.2.1 Flooding of Basin by Gravity

11.2.1.1. Capable of drawing water from the adjacent waterway at all tides

11.2.1.1.1. Invert of 48-inch intake pipes set at -8.33' (MLLW). Lowest observed tide is -3.35'. Pipes are submerged during all tides.

11.2.1.1.2. 48-inch pipes allow the basin to fill in a relatively short period of time for all planned cycles. Time is dependent upon tides and pontoons present in basin. These large diameter pipes also provide greater filling flexibility for unknown future operations.

11.2.1.2. Gravity flooding occurs through both pipes. Once water surface levels inside and outside have equalized basin gate can be removed. Flooding will continue through the opened gate. Pumping is not necessary to achieve float out water surface elevations. Flood times are dependent upon float out cycle and tides. Flooding of basin via 48" pipes and through the main gate can be achieved to allow sufficient float out time for each of the six planned cycles for the majority of predicted tides. O&M manual will provide more information.

11.2.1.2.1. All tidal data used is 2012 predicted data from NOAA Tide Station 9441187, Aberdeen, WA

11.2.1.2.2. Float out is assumed to take no longer than approximately 6 hours from when the pontoons lift off until they are completely loaded out.

11.2.1.2.3. Gate removal is assumed to take approximately 3 hours from the time the basin water surface equilibrates with the channel water surface and the gate removal process is initiated until the basin gate is clear and ready for float out.

11.2.1.3. Intake fish screen sized to accommodate a 200 cfs uniform flow. Sluice gate opening will be adjusted to control the flow through the fish screen. As the tide falls and basin fills the sluice gate will be opened further to maintain a uniform flow rate of 200 cfs or less.

11.2.1.3.1. Screen is sized per the WDFW screening requirements technical assistance document. Screen assumed for design is profile bar suitable for screening out fry.

11.2.2 Pump Assisted Emptying of Basin

11.2.3.1 Pumps are utilized after main gate is replaced. Pumps are capable of emptying basin in approximately 12 hours from MLLW. This time will vary due to the water levels not being at MLLW when the basin gate is placed. A 12 hour emptying time provides a balance between time, space for pumps/size of pumps and availability of rental pumps. No time allotment for fish handling has been included. O&M manual will provide more information.

11.2.3.2 Pump screen sized to accommodate a 27000 gpm flow. This will accommodate six nominal 4000 gpm pumps at all expected pumping heads.

11.2.3.2.1 Screen is sized per the WDFW screening requirements technical assistance document. Screen assumed for design is profile bar suitable for screening out fry.

11.2.4 Capable of Gathering and Removing Fish

11.2.4.2 A fish box is provided to meet ESA requirements as part of RFP Commitments

11.2.4.2.1 Fish box is used to transport fish from the basin to the channel. Fish are herded into the box where a crane will then be used to move the box from the basin to the channel to return the fish to Grays Harbor.

11.2.4.2.2 Fish box size is a result of conversations and meetings with WDFW

11.2.4.2.3 Fish box will have a screened cover to prevent fish from jumping out while being returned to the channel and will be aerated to maintain the dissolved oxygen levels prescribed by WDFW in the HPA.

11.2.4.2.3.1 It is assumed that the DO levels will be between 5-7ppm and can be maintained by the use of airstones or other air diffusers. Issuance of HPA may alter this portion of the design.

11.3 Proposal Assumptions and Options Considered

11.3.1 Assumptions

- 11.3.1.1. 0.4 fps approach velocity with no sweeping velocity is compliant at all screens including pump screens
- 11.3.1.2. Velocity of water in the channel near the fish screen box is negligible or may be ignored
- 11.3.1.3. 4500 gpm per pump is the maximum pumping rate
- 11.3.1.4. Tides affect the flooding and emptying times and constrain the time allowed for float outs.
- 11.3.1.5. HPA will not change fish screening requirements

11.3.2 Options Considered

- 11.3.2.1. Pumps only

11.4 Engineering Concepts Explored Since the Proposal

PART 12.0 BASIS OF DESIGN-GEOTECHNICAL CONSTRUCTION

12.1 Proposal Assumptions and Options Considered –

The Design-Build team evaluated several options for constructing the proposed pontoon casting facility (PCF) at the Aberdeen Log Yard in Aberdeen, Washington. After several iterations, the two options studied in detail included an at-grade scheme and a basin located at about elevation -9 feet. The at-grade scheme included constructing a pile-supported concrete slab to facilitate pontoon construction, and overwater piers that would support a gantry crane that will move the constructed pontoons into the water upon completion. The basin scheme included designing an excavation with appropriate side slopes, a pile-supported concrete slab to facilitate pontoon construction, and a gate that would be removed to float the pontoons out of the facility. Upon completion of the initial pontoon construction and float out, the basin would be emptied, and the process repeated. Foundation recommendations were based on the

Request for Proposal (RFP) documents including the Geotechnical Data Report dated August 17, 2009, that included the results of subsurface explorations, in situ testing, and laboratory tests.

12.1.1 Casting Basin Slab Support

12.1.1.1. The Design-Build Team considered two options for the PCF including constructing a facility at-grade (top of concrete slab elevation +15 to +20 feet), as well as a basin facility with a top of slab located at about elevation -9 feet. The at-grade solution would include placement of pontoons into the water with a gantry crane that would traverse along a pile-supported pier. The basin facility would require a major excavation and would be flooded upon completion of the pontoon construction to float the units into the water.

As part of the preliminary studies, various slab thicknesses and support options were evaluated that would enable the pontoons to be constructed within the specified pontoon fabricating tolerances. In general, for a 75-foot-wide by 360-foot-long pontoon, the maximum differential settlement relative to the center of the pontoon is 0.75 inch at its ends (0.75 inch in 180 feet) and 0.25 inch at the outside center edge of the pontoon (0.25 inch in 37.5 feet). These tolerances were specified in the Request for Proposal.

The differential settlement of the support slab were estimated using a three-dimensional model and FLAC 3D, Version 3.1, a computer program developed by Itasca Consulting Group, Minneapolis, Minnesota. FLAC 3D is a three-dimensional finite difference program that is designed for modeling continuous materials such as soil. FLAC 3D was used to calculate the total stress changes and resulting settlement as a result of pontoon construction loading.

In the analyses, the pontoon loads (750 pounds per square foot [psf]) were placed instantaneously over the entire pontoon plan area, rather than constructed over a period of months. Although a detailed pontoon construction sequence could be incorporated into the model to better estimate the differential settlements of the support slab and pontoon, this was not accomplished, as such an approach would likely result in construction-induced stresses into the pontoon slabs and walls, which is generally prohibited by the RFP.

The stiffness and strength of the support slab were incorporated into the FLAC model. Cracked and uncracked slab section properties were provided for slab thicknesses varying between 1 and 5 feet.

The results of the explorations and laboratory testing were used to develop the subsurface soil profile for analysis. Because ground failure was not considered in this portion of our study, the Isotropic-Elastic (IE) model was selected, which does not allow shear yielding. The IE model treats a material as purely elastic with the same elastic properties in all directions. That is, the material has the same linear elastic behavior in each of the x-, y-, and z-directions. The properties required for the IE model, as implemented in FLAC 3D, include unit weight and elastic properties.

A series of parametric studies were performed to evaluate the feasibility of constructing a relatively thick slab to support the pontoon construction without structural support. The following analyses were performed:

1. Construct a slab near the existing site grade with the surficial fill and timber debris in place.
2. Construct a slab near the existing site grade after excavation and replacement of the surficial fill/timber debris with granular fill.
3. Construct a slab either near the existing site grade or at elevation -8 feet after the soil had been improved to a depth of about 50 feet below the slab using various ground improvement options.

   The results of the analyses are summarized in Tables 12.1 through 12.3. In general, the analyses indicated that to satisfy the differential settlement tolerances specified for the pontoons, it is not possible to construct the slabs on the existing ground or at elevation -8 feet without improving the ground in the upper 50 to 60 feet of soil.

12.1.1.2. Ground Improvement Options

Considering the very soft, sensitive nature of the silt soil, potential ground improvement options considered include relatively closely spaced timber piles and/or some form of in-situ soil-cement such as jet-grouting or deep soil mixing. Based on the equivalent modulus required for the upper soil to satisfy the allowable differential settlements, it was estimated that timber piles spaced 7 feet center-to-center driven to a tip elevation of about -65 feet would achieve the necessary ground improvement to enable the pontoons to be constructed within settlement tolerances. However, there is some risk of excessive differential settlements occurring as a result of ground settlements below the tips of the timber piles. The top of the very dense sand and gravel soil varies beneath the PCF. In addition, the nature and consistency of the soil also may vary. As a result, differential settlements could occur over the large pontoon area due to differential settlement of the soil beneath the timber piles. In this regard, structurally supporting the slab with steel pipe piles extending to the very dense sand and gravel layer may be a cost-effective alternative, as compared to the timber piles. If there is no cost savings with the timber pile "ground improvement" scheme, structurally supporting the slab with steel pipe piles extending to the very dense sand and gravel would provide a more positive solution, and less risk that the differential settlement tolerances would be exceeded. Potential in situ soil-cement options such as deep soil mixing were also evaluated. Based on preliminary evaluations, deep soil mixing was not evaluated further due to cost considerations.

Table 12.1 summarizes the results of analyses that evaluated the feasibility of constructing the slab at the existing grade, with and without removing the timber debris. Table 12.2 presents the results of the initial analyses performed to evaluate potential ground improvement schemes. The differential settlement results are further summarized in Figures 12.1 and 12.2. Table 12.3 presents the results of parametric studies after a suitable upper soil modulus was determined, assuming ground improvement.

If the ground is improved using timber piles spaced at 7 feet center-to-center, the piles could likely be driven followed by the placement of a geotextile and 3 feet of granular material. The concrete support slab could then be constructed atop the granular material surface, such that it would not be necessary to structurally connect the timber piles to the support slab. The granular material would also function as part of the passive drainage system required to maintain groundwater levels below the slab.

It would likely be difficult to drive the timber piles from the existing ground surface without encountering numerous obstructions. For a 7-foot center-to-center pile spacing, over 5,000 piles would be required to support the base slab of the PCF. Although the pile spacing could be adjusted periodically if an obstruction is encountered, it is likely that significant excavation would be required at numerous locations to successfully install the 5,000 piles. This closely spaced timber pile option would likely be a more efficient and successful option for the basin facility for the case where the pile installation would commence near elevation -8 feet after removal of the surficial fill and timber debris.

12.1.1.3. Pile Supported Option and Gantry Crane Foundation

In lieu of the ground improvement option, the Design-Build Team will consider structurally supporting the base slab with driven pile foundations. Pile types that are currently being considered include: 18-, 24-, and 30-inch steel pile piles driven closed-end; and 24-inch octagonal, prestressed concrete piles. The size and type of piles used would depend upon the required axial resistance, as well as which PCF option is selected. For a cost-effective slab, since debris is located at the site, it is recommended that steel pipe piles be used. To increase the likelihood of penetrating through the wood debris, it is recommend that the smallest pile diameter be used and that the piles be fitted with a conical point rather than a flat-bottom plate. Although the conical tip would help penetrate through the wood debris, some large timber obstructions will likely be encountered during initial driving (as noted in the test pit photographs included in Appendix A of the August 2009 Conceptual Geotechnical Report) that will require excavations to remove the obstructions prior to reaching the required pile tip elevation.

Because of the timber obstructions and the very soft soil underlying them, prestressed concrete piles are not recommended to be installed without removing the timber debris prior to initial driving, if the piles are installed at the existing ground surface. The debris and soft soil could likely result in high tensile stresses, causing pile damage prior to reaching the required tip elevation. Prestressed concrete piles could, however, be considered for the overwater piers and the basin option if the piles are installed after the basin is excavated and the surficial debris is removed. However, that the 24-inch prestressed concrete piles will weigh about three to four times more than a comparable steel pipe pile as noted above.

The required pile axial resistance based on service loads may vary between 400 and 900 kips depending upon pile spacing, resulting in a required factored axial resistance of about 600 to 1,300 kips. Greater loads were considered for the 30-inch-diameter steel pipe piles that may be used to support the overwater piers.

Fill may be placed to raise the site for the at-grade PCF scheme. Depending upon the top-of-slab elevation (+18 to +20 feet MLLW), 1 to 5 feet of fill may be required to raise the site to accommodate the slab construction. The addition of fill at the site will cause ground settlements that will result in downdrag loads acting on the pile foundations. It would be more appropriate to establish the PCF slab elevations to minimize fill placement at the site.

Ultimate pile resistances were evaluated for the various pile types being considered at the site. In addition to evaluating the ultimate resistance based on the soil conditions encountered at the site, Wave Equation Analysis of Piles (WEAP) was performed to evaluate potential pile wall thicknesses and pile-driving hammers. The results of the pile resistance analyses are summarized in Table 12.4. As noted, greater ultimate resistances could be achieved for the 24-inch-diameter pile if the pile wall thickness is increased from 0.5 to 0.75 inch.

Estimated vertical spring constants, k, for each pile type considered are summarized in Table 12.5. Recommended soil parameters for static, lateral analysis of piles using computer programs LPILE Plus and DFSAP are presented in Table 12.6.

12.1.1.4. Dewatering (Temporary)

To construct the excavation, the area should be temporarily dewatered as the excavation proceeds. Two dewatering schemes were proposed. Scheme A includes installing trench drains and vacuum extraction wells around the perimeter of the excavation. Scheme B includes installing deeper dewatering wells throughout the excavation limits.

The excavation/dewatering sequence for Scheme A is presented in Drawing CB4A prepared for the project, as summarized below:

1. Excavate the basin area to at least 2 feet above the existing groundwater level of the site. The depth of this initial excavation could vary between about 1 and 15 feet, depending upon the proximity of the shoreline.

2. Install vacuum extraction wells near the toe of the excavated slope around the perimeter of the excavation. The wells should be spaced about 20 feet center-to-center and should extend to about elevation -9 feet. Connect the wells to a header pipe and operate the system.

3. While well installation is ongoing, excavate trench drains perpendicular to the river, extending from one end of the basin to the other. The trench drains should be spaced about 50 feet, center-to-center, with a drain located at the toe of the excavated slope. The base of the trench drains should extend to a depth of about 6 feet below the bottom of the future PCF concrete slab. Install a subdrain pipe and backfill the trench drains

with free-draining sand and gravel to the bottom of the future PCF concrete slab. Install sump pumps at each end of the trench drains and operate the system to lower the groundwater level in the vicinity of the excavation.

4. Continue excavation of the basin area. The excavating and operating on the native silt soil will be difficult. It will likely require that 2 to 3 feet of quarry spalls be placed and pushed into the silt material to provide a working surface for tracked vehicles. If possible, the working surface would better support equipment, if a geotextile is placed prior to placement of the quarry spalls.

5. Upon completion of the excavation to about 3 feet below the base of the future slab, install the permanent drainage system which includes placing 2 to 3 feet of free-draining sand and gravel atop a geotextile placed on the quarry spalls. Perforated pipes should be spaced a minimum 50 feet center-to-center at the base of the drainage sand and gravel layer. The pipes should be routed to the permanent sump system. Based on discussions with Kiewit-General, the permanent drainage system does not require the presence of the quarry spalls or geotextile. In addition, we understand that they would like to limit the thickness of the sand and gravel layer. As a result, the thickness of the free-draining sand and gravel layer was reduced to 2 feet, and the spacing of the perforated subdrain pipes was also reduced to about 20 feet. This information is presented in Drawings CB4D, CB4E, and CB4F.

6. Place the free-draining granular material along the excavated slope.

We understand that this Scheme A will not be used. As an alternative to Scheme A, temporary dewatering Scheme B was developed. In lieu of installing the trench drains, which are anticipated to be costly and difficult to construct, deep dewatering wells were evaluated. It appears that the site can be temporarily dewatered by installing and operating conventional dewatering wells to about elevation -50 feet at a spacing of about 70 to 80 feet center-to-center. As the site is excavated and the permanent system is installed and operated, the dewatering wells could be decommissioned. Considering that the basin will be flooded and unwatered several times, it is recommended that selected wells be maintained and extended through the PCF base slab. In the event that groundwater levels are not lowered in a timely manner as the basin is unwatered after a flooding sequence, these dewatering wells could be operated to speed up groundwater level drawdown below the PCF base slab. After the excavation is completed to subgrade level, the permanent dewatering system can be installed as indicated in steps 4 through 6 for Scheme A. Scheme B is summarized in Drawings CB4C and CB5.

In addition to the dewatering requirements specified in Schemes A and B, the Contractor should be prepared to install and operate sumps at low points excavated throughout, as the mass excavation proceeds. In several of the test pits excavated at the site, groundwater seepage was observed near the fill/native soil contact. It is anticipated that depending on the nature of the fill and the proximity of the river, groundwater flow volumes will vary throughout the site. The Contractor should be prepared to manage such conditions by utilizing flatter slopes in local areas, by installing local sumping measures, by installing sand/gravel/rock filters, and/or in the more extreme event, by installing sheet piles to limit groundwater flow and soil movement.

Based on typical sections developed for the temporary excavation that will be required to construct the gate foundation, it appears that the crest of a 3H:1V slope is located about 80 to 120 feet away from the shoreline. Given this distance, a temporary excavation with 3H:1V side slopes is feasible provided that the area is adequately dewatered. It is recommended that the Contractor install four dewatering wells, as shown in Drawing CB10B, to lower the groundwater level in the excavation areas. It is also recommend that the Contractor include contingencies in the estimate to install a sheet pile cutoff wall around portions of the perimeter of the excavation to limit groundwater inflow depending on the actual subsurface conditions encountered in the slope area.

12.1.1.5. Dewatering (Permanent)

Based on the subsurface conditions encountered near the basin subgrade level, the basin can likely be dewatered during the pontoon construction using the recommended sand and gravel layer placed below the permanent basin slab, as described above. The perforated drains located in the sand and gravel layer should be routed to the sump locations installed around the perimeter of the basin. The sumps should be continuously pumped to maintain lowered groundwater levels so that uplift pressures do not act on the base of the PCF slab. In addition, as indicated above, selected dewatering wells installed for Scheme B be carried through the PCF slab to provide additional dewatering capacity in the event that groundwater levels are not lowered in a timely manner during the unwatering cycle.

Potential groundwater discharge rates of the PCF permanent dewatering system were evaluated by constructing a steady-state, three-dimensional numerical groundwater flow model. The U.S. Geological Survey computer program MODFLOW (McDonald and Harbaugh, 1988) was used, which is part of the Groundwater vistas (version 3.15) groundwater modeling package (Rumbaugh and Rumbaugh, 2001). The model for the PCF permanent dewatering system was constructed based on the stratigraphic sequence observed in the soil borings and test pits, as well as data from the aquifer test conducted by WSDOT in July 2009. The evaluation includes the following assumptions:

- Initial groundwater levels between elevations 6.5 and 8 feet based on groundwater elevations observed in the monitoring wells
- Hydraulic conductivity: 2 to 10 feet per day
- Storage coefficient: 0.1
- Drawdown criteria: up to 20 feet.

The groundwater discharge from the permanent dewatering system could range from about 100 to 200 gallons per minute. Flow rates should decrease over time as the saturated thickness of the water-bearing soil decreases as a result of dewatering. If subsurface conditions differ during excavation, the groundwater discharge rates may be greater.

The results of the drawdown analyses are presented in Figures 12.3 and 12.4, for 75 day and 1 year of pumping, respectively, and show contours of groundwater drawdown outside the excavation. Based on the subsurface conditions encountered at the site, it is estimated that for 5 feet of continuous groundwater drawdown, the ground surface could settle about 1.5 to 2 inches. The magnitude of the settlement is generally proportional to the magnitude of the drawdown and the consolidation properties of the underlying soil. Note that Section 5.10 of the Geotechnical Baseline Report dated September 16, 2009, prepared by Landau Associates, indicates that 5 feet of fill placed at the adjacent wastewater treatment plant site during original construction in 1959 caused about 14 to 16 inches of primary settlement. Several inches of secondary settlement occurred in the 10 to 20 years following construction. (This information was presented in a report prepared by Shannon & Wilson in 2001 for proposed plant site modifications.) If these settlement data are accurate, the above estimated settlement of 1.5 to 2 inches for 5 feet of groundwater drawdown would increase to about 7 to 8 inches of settlement.

It is recommended that the foundation conditions of the adjacent Aberdeen Wastewater Treatment Plant facilities be evaluated to determine the potential impacts of ground settlement on these existing structures. Furthermore, it is recommended that settlements as a result of groundwater level drawdown be considered in the design of utilities and other structures located throughout the site. An instrumentation monitoring program including the installation of settlement monitoring points and groundwater observation wells should be implemented in the project vicinity with particular emphasis placed on adjacent existing structures.

Seepage analyses were conducted to evaluate water movement below the PCF gate and sheet pile cutoff wall to estimate exit gradients at the interface between the drainage layer and the underlying native soil. Potential groundwater seepage conditions were evaluated at the PCF gate and cutoff wall by constructing a groundwater flow model using the two-dimensional, finite-element seepage analysis program SEEP/W 2007, which is part of the GeoStudio 2007 software package developed by Geo-Slope International (2007).

The steady-state seepage models were developed based on soil and groundwater data collected during previous explorations by WSDOT (Landau, 2009). The evaluation includes the following assumptions:

- Initial groundwater levels between elevations 6.5 and 8 feet based on groundwater elevations observed in the monitoring wells
- Chehalis River stage elevation 5 feet
- Drainage layer head elevation -11.5 feet
- Surface elevation -13 feet on river-side of cutoff wall
- Sheet pile cutoff wall depths varying between 20, 30 and 40 feet
- Saturated soil hydraulic conductivity: drainage sand: 285 feet per day, trace to slightly silty sand: 140 feet per day, silty sand: 3 feet per day, silt: 0.03 foot per day

Based on the results of these analyses, it is recommended that the tip of the sheet pile cutoff wall extend to elevation -40 feet below the gate structure. This elevation should transition to -45 feet in areas where soil is located along the outboard side, as noted above.

12.1.1.6. Geotechnical Baseline Report - A Geotechnical baseline Report (GBR) dated September 16, 2009, was prepared for the project. The following items are reproduced from the GBR (items underlined were reproduced verbatim):

12.1.1.6.1. Section 5.3, Existing Structures – As mentioned in Section 4.4.1 of the GBR, numerous concrete slabs/foundations were observed at the site with pile foundations expected under some of these foundations. Figure 10 (included in Appendix A of the August 2009 Conceptual Geotechnical Report) shows generalized areas where slabs and pile foundations related to previous site development are expected to be present at the site. This figure was prepared based on historical maps included in Appendix G7 of the RFP. As a baseline assumption, the Design-Builder shall assume the presence of concrete slabs supported on 40-foot timber piles within the hatched areas shown in Figure 10. The Design-Builder shall assume that the concrete slabs are in an intact condition and, therefore, will need to be broken by mechanical means. Timber piles are observed protruding out of the offshore sediments along the shoreline and in the areas of the proposed casting basin channel (refer to Figure 10 of the August 2009 Conceptual Geotechnical Report) and the Design-Builders shall assume these piles will be encountered in any shoreline or offshore excavation or dredging.

12.1.1.6.2. Section 5.4, Perimeter Walls – The construction of some types of perimeter walls (e.g., cantilever walls, mechanically stabilized earth [MSE] walls) will require mass excavation. The Design-Builder shall assume that wood debris, logs, buried structures, and other objects will be encountered during the excavation required for these types of walls. In the case of using sheet pile walls, the Design-Builder shall assume as a baseline condition that driving and/or vibrating sheet piles along 85 percent of the perimeter alignment is not feasible (without pre-excavation) because of the presence of existing structures (refer to Figure 10), logs, wood debris, and other objects within the fill layer. Within Units 1 through 3, the Design-Builder shall assume as a baseline condition that logs (in Units 1 and 2) and boulders (in Unit 3) will be encountered along 2 percent of the casting basin perimeter, where driving and/or vibrating sheet piles are not feasible.

12.1.1.6.3. Section 5.6.1, Piles – Driven precast concrete, timber piles, and cast-in-place concrete (installed within steel casing) were considered in the development of the conceptual design. As a baseline assumption, the Design-Builder shall assume that the pile installation will encounter logs and other objects at any depth within Units 1 through 3 that prevent the advancement of the pile to the design bearing layer or the design tip elevation at 2 percent of the pile locations.

12.1.1.6.4. Section 5.7, Gate Structure – Driven piles are expected to be used to support the gate structure. Shallow foundations may not be feasible based on anticipated high lateral hydrostatic pressure and given the expected sensitivity of the gate to settlement. The issues and the baseline assumption regarding the potential to encounter logs, wood debris, boulders, etc., listed under Section 5.6.1 are also applicable to the gate foundations.

12.1.1.6.5. Section 5.8, Batch Plant and Crane Foundations – As a baseline assumption for the batch plant and the crane foundations, the Design-Builder shall assume that the pile installation will encounter refusal prior to reaching the design bearing layer or the design tip elevation due to the presence of concrete slabs, logs, and other objects within the fill layer at 50 percent of the pile locations. Within Units 1 and 3, the baseline assumption shall be the presence of logs, boulders, and other objects at 2 percent of the pile locations.

12.1.1.6.6. Section 5.11, Site Grading and Earthwork Considerations – Mass excavation will be required within the footprint of the casting basin and the entrance channel area. The Design-Builder shall assume that logs, buried structures, wood debris and other objects will be encountered during the excavation. Site grading will require cutting and filling across the site to achieve the final site grade. In areas receiving fill, the presence of the compressible fill and Units 1 and 2 will cause significant settlement to occur, as described in Section 5.10. Some areas may need to be overexcavated to accommodate the placement of the pavement section and other structures. The Design-Builder shall assume that overexcavation materials will include concrete slabs, piles, wood debris, railroad ties, and logs. In the areas of the existing structures shown in Figure 10 of the August 2009 Conceptual Geotechnical Report, the Design-Builder shall assume the conditions presented in Section 5.3 of the GBR.

The existing fill to be excavated may not be suitable for re-use because of the extensive wood debris at the site. Additionally, most of the fill is sensitive to moisture because of the relatively high fines content. As such, the Design-Builder shall assume that the existing fill is not suitable for re-use as structural fill. Additionally, Unit 1 shall be assumed to be not suitable as structural fill because of its composition and moisture conditions (wet, massive silt).

12.1.1.6.7. Section 5.14, Utilities – Excavation for utilities would be mainly through the existing site fill, which includes logs, existing structures, and other forms of debris that may impact the installation progress and the stability of the utility trenches. The removal of hard objects, such as concrete slabs, may require special equipment. As a baseline assumption, the Design-Builder shall assume the presence of existing structures, as described in Section 5.3 of the GBR. Additionally, in all areas, including the areas where concrete slabs are present, the Design-Builder shall assume that a log will be encountered in every 100 lineal feet of utility trenches and that the removal of this log cannot be achieved with conventional trenching methods. Logs, wood debris, and other objects present at the site shall be assumed to be removed with conventional, heavy excavating equipment typically used for trench methods.

Regarding utility support, the Design-Builder shall assume that along 75 percent of the utility trenches up to 10 feet in depth, unsuitable materials for proper foundation support are present. For utility trenches deeper than 10 feet, the Design-Builder shall assume that along 50 percent of the utility trenches, unsuitable materials for proper foundation support are present.

12.2 Construction Methods – General

As noted in Section 12.3, the site is underlain by fill with significant timber debris, followed by very soft to medium stiff silt of medium to high plasticity to an elevation of about -60 to -70 feet MLLW. Based on the results of vane shear testing, the silt soil appears to be highly sensitive such that they will exhibit very low strengths upon disturbance. The silt soil also exhibits relatively high natural water content. The exploration logs and the natural water contents of samples retrieved from the explorations were reviewed. The depth to where the natural water content of the soil is less than about 80 percent was recorded. This depth range varies between about 0 and 57 feet, corresponding to about elevation +17 to -46 feet, with an average elevation equal to about -4 feet.

Silt soil with high natural water contents are difficult to traverse and will result in difficult access and trafficability of heavy construction equipment. Tracked vehicles will likely be required for excavation and removal of spoils. Temporary roadways constructed with 2 to 3 feet of quarry spalls pushed into the underlying soft soil and ahead of the equipment will generally be necessary at this project site. Placement of heavy geotextiles with the quarry spalls would improve the roadway performance. Placement of the geogrids below the quarry spalls would generally provide a more stable roadway than if the geogrids are placed above the spalls. However, placement of the geogrids prior to the spalls would create more difficult constructability issues.

The significant timber debris in the fill material combined with the very soft soil will also complicate pile driving. If the piles are driven from the existing ground surface, numerous obstructions will likely be encountered. This would likely require local excavations at each pile location. In addition, if prestressed concrete piles are selected, hard driving conditions through the rubble, followed by very soft driving may result in tensile stresses in the pile during installation, resulting in pile damage. If the rubble fill is excavated prior to pile driving, the underlying soft soil would be difficult to traverse. Prestressed concrete piles weigh about three to four times more than a comparable steel pipe pile, such that construction of a more significant roadway atop the soft soil would be required to support the rigging and pile installation equipment. In addition, the required length of the piles would generally require that the pile be spliced. Considering the soft soil and the weight of the pile, there are concerns that the first section of the pile would "run", greatly complicating the installation and splicing procedure. In general, these concerns would be less for closed-end steel pipe piles. Axial resistance and pile driving recommendations were developed for several different pile types. These recommendations were incorporated into the slab design to develop the most cost-effective pile-slab system that would perform appropriately.

Groundwater conditions will require the subsurface soil to be dewatered prior to, or during the excavation. Based on review of the soil conditions at the site and the results of the pumping tests performed by WSDOT, closely spaced wells and a relatively long period of time would generally be required to lower the groundwater levels at the site. Additional pumping tests are proposed at the site to better evaluate dewatering alternatives. For the proposal phase, two dewatering schemes were developed including trench drains and dewatering wells. The dewatering well option was selected for the project. The scheme presented will be reanalyzed to determine if a more cost-effective, and more constructible dewatering option can be developed

12.3 Subsurface Conditions and Construction Considerations

12.3.1 Site Description -

The site is located on the east side of East Terminal Way at Cow Point in an industrial area of Aberdeen, Washington. The ground surface elevation of the site generally varies from about 4 feet mean lower low water (MLLW) near the shoreline, to about 16 feet MLLW. The site, in general, is relatively flat with numerous log stacks and occasional stockpiles of wood debris and soil. Log stacks were surveyed as high at 30 feet MLLW. Several old concrete pads in very poor condition are scattered throughout the site. Dirt or gravel roads exist throughout the site, and most of the roads are in relatively poor condition. A distressed pile-supported concrete slab was observed at the site. The tops of the piles have generally punched through the slab, and the slab appeared to have settled at least 6 to 12 inches.

The City of Aberdeen Wastewater Treatment Plant is located immediately east of the proposed project site. A clarifier is located adjacent to the southeast corner of the property. Port of Grays Harbor Terminal 4 is located west of the site.

12.3.2 Subsurface Soil Conditions

A site plan and several subsurface cross sections were included in the Geotechnical Data Report prepared by WSDOT. The results of the explorations located within the general limits of the proposed casting basin were reviewed. A summary of selected soil and groundwater conditions noted in the explorations is presented in Table 2.7.

In general, the subsurface conditions consist of fill with numerous wood debris and occasional concrete debris to a depth of about 10 to 15 feet below the existing ground surface. As indicated in Table 2.7, the maximum depth where wood debris was noted on the exploration logs. In some explorations, the thickness of the wood debris appeared to be extensive, while in others it may be limited to 6 to 12 inches, buried at significant depths. The depth of wood debris noted in the logs varied between 0 and 23.5 feet, with an average depth of about 11 to 12 feet. A review of the test pit photographs included in the RFP provides additional insight into the volume of wood debris and the magnitude of groundwater seepage that may be encountered across the site.

Very soft to medium stiff silt of medium to high plasticity underlies the fill to an elevation of about -60 to -70 feet MLLW. Based on the results of Atterberg limit testing, this soil does not appear to be liquefiable, but they would likely experience some strength loss under earthquake loading. In addition, based on the results of vane shear testing, the silt soil appears to be highly sensitive such that they will exhibit very low strengths upon disturbance. The silt soil immediately below the fill exhibits relatively high natural water content. The exploration logs and the natural water contents of samples retrieved from the explorations were reviewed. The depth to where the natural water content of the soil is less than about 80 percent was recorded. This depth range varies between about 0 and 57 feet, corresponding to about elevation +17 to -46 feet, with an average elevation equal to about -4 feet.

Very loose to medium dense, silty sand and medium stiff to stiff silt underlie the surficial silt encountered at the site to about elevation -90 to -110 feet (MLLW). Based on a review of the subsurface profiles included in the RFP, the silty sand does not appear to be continuous across the site. The silty sand is likely susceptible to liquefaction under earthquake loading.

Dense to very dense sand and gravel was encountered below an elevation of about -90 to -110 feet.

12.3.3 Groundwater Conditions

Groundwater conditions at the site consist of a water level at approximately elevation +5 to +11 feet MLLW (average elevation +7.3 feet—see Table 2.7), about 3 to 16.5 below ground surface (average depth of 8.3 feet—see Table 2.7). Groundwater levels are expected to be influenced by tidal fluctuations.

12.4 Final Design Methodology

12.4.1 Launch Channel and Dolphin Piles. The launch channel slope was evaluated for an inclination of 3 horizontal to 1 vertical (3H:1V) with a rock blanket and for an exposed native slope with an inclination of 5H:1V. Batter and plumb dolphin piles will be installed in the channel for a fender system that would be used to guide the pontoons as they are launched from the PCF. Dolphin piles would be constructed with 24-inch-diameter by ½-inch-thick wall open-end steel pipe piles. The turning dolphins would be constructed with battered 30-inch-diameter by ⅜-inch-thick wall open-end steel pipe piles, and the plumb piles would be constructed with 60-inch-diameter by ¾-inch-thick wall open-end steel pipe piles. The piles will be founded in the dense to very dense sand and gravel layer.

12.4.2 Gate Structure. The construction sequence of the launch channel/gate area will be managed such that the construction of the casting basin gate area will be constructed in the dry within the casting basin excavation, separated from the Chehalis River by a temporary sheet pile cutoff wall at the berm along the riverbank. Temporary dewatering wells will be installed so the excavation can be accomplished in the dry. The existing berm will remain in place until gate construction is complete.

The gate structure consists of a sill, jamb, and bulkhead walls. The gate will be held in place by the jambs and sill. The sill is the bottom submerged horizontal element that connects to the jambs. East and west jamb columns extend upward from the sill. Bulkhead walls extend east and west from the jambs and intersect the slope that continues upward from the casting basin slab elevation. The sill will be supported with 18-

by 3/8-inch-thick wall closed-end steel pipe piles. The jamb and bulkhead walls will be supported by 24- by 0.401-inch-thick wall open-end steel pipe piles. The piles will be founded in the dense to very dense sand and gravel layer.

The bulkhead walls would be constructed with concrete panels near the jamb and transition into steel sheet piles near the top of slope. A sheet pile cutoff wall will be constructed below the gate structure to control seepage. Lateral spreading displacement during the design ground motion has been provided to the structural engineer for design of the gate structure.

12.4.3 Casting Basin Excavation. Temporary dewatering wells will be installed so the casting basin excavation can be accomplished. Primary dewatering during construction will be accomplished with temporary dewatering wells and perimeter interceptor drains installed on both sides of the casting basin excavation. If necessary, the wells will be supplemented by temporary sump pumps placed in the excavation area. In general, the groundwater collected from the dewatering system will be re-infiltrated into the ground using an infiltration trench on the east side of the property. This re-infiltration of groundwater would reduce the potential for movement of the Aberdeen Wastewater Treatment Plant. Alternative backup methods for temporary water storage, prior to infiltration, could include diversion to on-site water storage areas.

12.4.4 Basin slab. The 18-inch-thick reinforced concrete basin slab will be supported by 18-inch-diameter driven closed-end steel pipe piles spaced on 16-foot centers. The top of slab elevation will be -9 feet. The steel pipe piles will be driven from the existing ground surface prior to excavation for the casting basin. The steel pipe piles will either be driven with a follower to the basin elevation, or they will be driven continuously and cut off at the basin elevation with a cutting tool prior to basin excavation. The piles will be founded in the dense to very dense sand and gravel layer.

During operation of the casting basin, groundwater will be collected from underdrains below the casting basin slab and groundwater cutoff trenches in the side slopes. The groundwater collected from the dewatering system will be re-infiltrated into the ground using an infiltration trench on the east side of the property.

12.4.5 Basin side slopes. Cut slopes with an inclination of 2.5H:1V will be excavated to reach the bottom of basin elevation from the existing ground surface. The final top-of-slope elevation will be between +17 and +18 feet. An approximately 4-foot-high toe wall will be constructed at the bottom of the basin side slope. To maintain local stability of the side slopes considering groundwater seepage, as well as flooding and unwatering of the basin during float-out, a 4-foot-thick layer of free-draining, graded, granular filter material consisting of 2 feet of sand and gravel and 2 feet of shot rock will be placed on the slope after excavation.

12.4.6 Gantry Crane Trestle. The gantry cranes that will be used for pontoon construction will be supported by the gantry crane trestle beams and 24-inch-diameter by 0.401-inch-thick wall steel pipe piles. Slope movements during the design ground motion has been provided to the structural engineer for design of the gantry crane trestle structure.

12.4.7 Stockpile. Most of the excavated material including organics from the casting basin will be placed on the onsite stockpile in the southwest portion of the site. The inclination of side slopes for the stockpile could range between 3H:1V and 4H:1V depending on soil consistency and placement methods.

12.4.8 Proposed Parking Lot. An asphalt parking lot will be located on the eastern side of the site. Site access would be obtained over asphalt entry roads from the northeast.

12.4.9 Precast Laydown Area and General Work Area. Pontoon construction will utilize both precast panels and cast-in-place concrete. The precast concrete panels will be fabricated in the precast laydown areas on the east and west sides of the casting basin. The precast laydown and general work areas would consist of gravel pavement. Construction equipment including cranes, loaders, forklifts, and trucks will be utilizing the gravel areas during construction of the PCF. The equipment axle loads and other specifications were provided by KG. Based on discussions with KG, we used an axle load of 64 kips from the Hyster

H300HD forklift as the design axle load. KG provided two values for the number of repetitions for Hyster forklift axle load: 1,500 and 11,000, to represent low and high volume trafficked areas, respectively.

12.4.10 Pipe protection slab. A pipe protection slab will be placed over the existing culvert at the west construction entrance. The pipe protection slab would be supported by spread footings.

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TABLE 12.1
SUMMARY OF FLAC3D
SETTLEMENT RESULTS

	Subsurface Profile[2] [E_{s1}-E_{s2}] (psi)	Concrete Slab Thickness (ft)	Center Settlement (in)	Left Edge Settlement (in)	Top Edge Settlement (in)	x Moment (k-in)	y Moment (k-in)	xy Moment (k-in)	Critical Moment[3] (k-in)
Uncracked Section	200-1,000	1	7.8	4.2	3.6	216	276	84	137
		2	8.3	5.4	3.6	648	540	204	546
		3	8.3	6.0	3.6	1,200	648	300	1,229
		4	8.4	7.2	3.6	1,920	744	360	2,184
		5	8.4	7.2	4.2	2,880	816	396	
	900-900	1	4.8	3.0	3.0	108	156	40	137
		2	5.0	3.6	3.0	372	456	88	546
		3	5.2	3.6	3.0	780	636	168	1,229
		4	5.2	3.6	3.0	1,320	768	240	2,184
		5	5.3	4.2	3.0	2,040	864	312	
Cracked Section	200-1,000	1	7.7	4.8	3.6	180	216	68	137
		2	8.2	4.8	3.6	540	504	180	546
		3	8.3	6.0	3.6	1,044	636	324	1,229
		4	8.4	6.0	3.6	1,680	732	396	2,184
		5	8.4	6.0	3.6	2,400	816	468	
	900-900	1	4.8	3.0	3.0	107	156	40	137
		2	5.0	3.6	2.4	372	116	114	546
		3	5.2	3.0	2.4	780	636	204	1,229
		4	5.2	3.6	2.4	1,320	768	288	2,184
		5	5.3	3.6	2.4	2,040	864	396	

Notes:
1. We performed the analyses using the computer program FLAC 3D (Itasca, 2008).
2. The analyses assumed two layers of elastic soil with elastic moduli E_{s1} and E_{s2}. Layer 1 was 14 feet thick, and Layer 2 was 86 feet thick.
3. The concrete slab properties and critical moment were provided by HNTB on August 25, 2009.
4. The analysis assumes that the pontoon loads are applied instantaneously. Additional analyses will be needed to evaluate the construction sequence loading.
5. A schematic of the concrete pad geometry (plan view) is shown below:
ft - feet; k-in = kips per inch; psi = pounds per square inch; psf - pounds per square foot

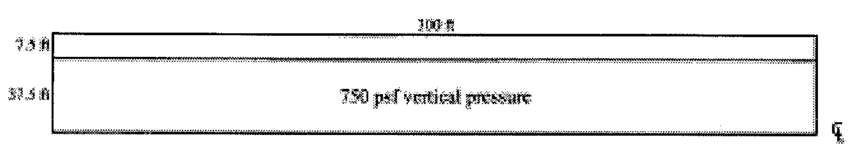

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TABLE 12.2
SUMMARY OF FLAC3D RESULTS
TESTING GROUND IMPROVEMENT

Es 0 to 50 feet (psi)	Es 50 to 90 feet (psi)	Center Settlement (in)	Left Settlement (in)	Top Settlement (in)	Center-Left Differential Settlement (in)	Center-Top Differential Settlement (in)
900	900	5.3	3.6	3.2	1.7	2.1
1,800		3.6	2.3	2.4	1.3	1.2
2,500		3.0	1.9	2.0	1.1	1.0
4,000		2.6	1.6	1.7	1.1	1.0
8,000		2.1	1.3	1.6	0.8	0.5
16,000		1.8	1.1	1.4	0.7	0.4
900	2,000	4.4	3.0	2.5	1.4	1.9
1,800		2.8	1.8	1.7	1.0	1.1
2,500		2.3	1.4	1.4	0.8	0.8
4,000		1.8	1.1	1.2	0.7	0.6
8,000		1.4	0.8	1.0	0.6	0.4
16,000		1.1	0.7	0.8	0.4	0.3
900	3,000	4.3	3.0	2.4	1.3	1.9
1,800		2.5	1.7	1.6	0.8	1.0
2,500		2.0	1.4	1.3	0.6	0.7
4,000		1.6	1.0	1.0	0.6	0.6
8,000		1.2	0.7	0.8	0.5	0.4
16,000		1.0	0.6	0.7	0.4	0.2
900	5,000	4.1	3.0	2.2	1.1	1.9
1,800		2.4	1.6	1.4	0.8	1.0
2,500		1.8	1.2	1.3	0.6	0.5
4,000		1.4	0.9	1.0	0.5	0.5
8,000		1.0	0.6	0.7	0.4	0.3
16,000		0.8	0.5	0.6	0.3	0.2

Notes:
1. We performed the analyses using the computer program FLAC 3D (Itasca, 2008).
2. We analyzed a 3-foot-thick, uncracked concrete slab. The slab properties and critical moment were provided by HNTB on August 25, 2009.
3. The analysis assumes that the pontoon loads are applied instantaneously. Additional analyses will be needed to evaluate the construction sequence loading.

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21-1-21190-001

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TABLE 12.3

SUMMARY OF FLAC3D RESULTS
TESTING GROUND IMPROVEMENT - UPPER SOIL Es = 16,000 psi

	Es 0 to 50 feet (psi)	Es 50 to 90 feet (psi)	Slab Thickness (ft)	Maximum x-Moment (k-in)	Maximum y-Moment (k-in)	Center Settlement (in)	Left Settlement (in)	Top Settlement (in)	Center-Left Differential Settlement (in)	Center-Top Differential Settlement (in)
Uncracked Section	16,000	2,000	1	24	34	1.1	0.6	0.9	*0.5*	*0.2*
	16,000	2,000	2	108	199	1.1	0.6	0.9	*0.5*	*0.2*
	16,000	2,000	3	266	434	1.1	0.7	0.9	*0.4*	0.3
	16,000	3,000	1	24	33	0.9	0.5	0.8	*0.4*	*0.2*
	16,000	3,000	2	105	187	0.9	0.5	0.8	*0.4*	*0.2*
	16,000	3,000	3	257	409	1.0	0.6	0.7	*0.4*	*0.2*
Cracked Section	16,000	2,000	1	17	23	1.1	0.6	0.9	*0.5*	*0.2*
	16,000	2,000	2	84	145	1.1	0.6	0.9	*0.5*	*0.2*
	16,000	2,000	3	196	367	1.1	0.6	0.9	*0.4*	*0.2*
	16,000	3,000	1	16	22	0.9	0.5	0.8	*0.4*	*0.2*
	16,000	3,000	2	82	137	0.9	0.5	0.8	*0.4*	*0.2*
	16,000	3,000	3	186	344	0.9	0.5	0.8	*0.4*	*0.2*

Notes:
1. We performed the analyses using the computer program FLAC 3D (Itasca, 2008).
2. The slab properties and critical moment were provided by HNTB on August 25, 2009.
3. The analysis assumes that the pontoon loads are applied instantaneously. Additional analyses will be needed to evaluate the construction sequence loading.

k-in = kips per inch
ft = feet
psi = pounds per square inch

21-1-21190-001-L2-Tables.xlsx

21-1-21190-001

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TABLE 12.4
SUMMARY OF PILE RECOMMENDATIONS

Pile Type	Pile Tip Condition	Pile Diameter (inch)	Pile Wall Thickness (inch)	Estimated Tip Elevation (ft, MLLW)	Estimated Ultimate Resistance (kips)	Estimated Downdrag Load (kips)	Recommended Pile Driving Hammer
Steel Pipe	Flat Bottom Plate	18	0.5	-125	1,000	0, See Note 1	D 62-22
Steel Pipe	Conical Tip	18	0.5	-135	1,100	0, See Note 1	D 62-22
Steel Pipe	Flat Bottom Plate	24	0.5	-125	1,500	0, See Note 1	D 80-23
Steel Pipe	Conical Tip	24	0.5	-135	1,500	0, See Note 1	D 80-23
Steel Pipe	Flat Bottom Plate	24	0.75	-125	1,600	0, See Note 1	D 80-23
Steel Pipe	Conical Tip	24	0.75	-135	1,700	0, See Note 1	D 80-23
Steel Pipe	Conical Tip	30	0.5	-130	2,000	0, See Note 1	D 100-13
Steel Pipe	Conical Tip	30	0.75	-135	2,400	0, See Note 1	D 100-13
Octagonal Concrete	-	24 (no splice)	-	-125	1,700	0, See Note 1	D 80-23
Octagonal Concrete	-	24 (with splice)	-	-125	1,300	0, See Note 1	D 80-23
Octagonal Concrete	-	16.5 (no splice)	-	-125	900	0, See Note 1	D 46-42
Octagonal Concrete	-	16.5 (with splice)	-	-125	750	0, See Note 1	D 46-42
Square Concrete	-	14 (no splice)	-	-125	800	0, See Note 1	D 46-42
Square Concrete	-	14 (with splice)	-	-125	650	0, See Note 1	D 46-42
Steel Pipe	Flat Bottom Plate	18	0.5	-125	900	120	D 62-22
Steel Pipe	Conical Tip	18	0.5	-135	1,000	120	D 62-22
Steel Pipe	Flat Bottom Plate	24	0.5	-125	1,350	170	D 80-23
Steel Pipe	Conical Tip	24	0.5	-135	1,350	170	D 80-23
Steel Pipe	Flat Bottom Plate	24	0.75	-125	1,450	170	D 80-23
Steel Pipe	Conical Tip	24	0.75	-135	1,550	170	D 80-23
Steel Pipe	Conical Tip	30	0.5	-130	1,800	200	D 100-13
Steel Pipe	Conical Tip	30	0.75	-135	2,200	200	D 100-13
Octagonal Concrete	-	24	-	-125	1,550	170	D 80-23

Notes:
1. These estimated ultimate resistance values assume that no fill is placed at the site to cause downdrag loads.
MLLW = mean lower low water

21-1-21190-001-L2-T5-T6.xlsx

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TABLE 12.5
SUMMARY OF VERTICAL SPRING CONSTANTS

Pile Type	Pile Diameter (inch)	Wall Thickness (inch)	Service Load Range (kip)	Estimated Vertical Spring Constant, K, (kip/inch)
Steel Pipe	18	0.5	450 to 650	600 to 650
Steel Pipe	24	0.5	450 to 650	850 to 900
Steel Pipe	24	0.5	650 to 800	750 to 850
Steel Pipe	24	0.75	450 to 650	1,250 to 1,350
Steel Pipe	24	0.75	650 to 800	1,150 to 1,250
Steel Pipe	30	0.75	650 to 800	1,500 to 1,650
Steel Pipe	30	0.75	800 to 1,000	1,450 to 1,600
Concrete	24	-	450 to 650	2,000 to 2,150
Concrete	24	-	650 to 800	1,850 to 2,000

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TABLE 12.7

SUMMARY OF SOIL BORINGS AT PONTOON CASTING FACILITY (PCF)

Boring Designation	Top Elevation (ft)	Depth to Water (ft)	Elevation to Water (ft)	Depth to Where Wood Debris is Noted (ft)	Depth to Where Water Content of Soil is Less Than 80% (ft)	Depth to Top of Dense Soil (ft)	Elevation to Top of Dense Soil (ft)	Depth of Consolidation Tests (ft)	Elevation Consolidation Test (ft)	Consolidation Test Soil ID	Initial Soil Wet Unit Weight (pcf)
H-1-08	16.2	5	11.2	20	23	106.5	-90.3				
H-7-09	16.3	Not Measured		12	18	124.5	-108.2	20	-3.7	OH	102.8
	16.3							109	-92.7	ML	110.8
H-6P-09	23.0	16.5	6.5	16	18	129	-106.0	27	-4.0	OH	106.1
H-05P-09	13.8	7.5	6.3	6.5	0	104	-90.2	19	-5.2	OH	107.4
H-03P-09	13.4	3	10.4	11.5	0	111	-97.6				
H-47-09	16.2	Not Measured		4	6	NE	N/A	17	-0.8	OH	100.7
H-17P-09	16.0	10	6.0	8	6	115.5	-99.5				
H-16P-09	17.2	Not Measured		6	0	107	-89.8	93	-75.8	CH	104
H-15P-09	14.7	7	7.7	11.5	11.5	112.5	-97.8	89	-74.3	OH	105
H-14P-09	11.8	5.5	6.3	19	11	119	-107.2	82	-70.2	ML	108.3
H-13P-09	12.5	6.5	6.0	23	23	127	-114.5				
H-28-09	10.7	Not Measured		8	52	107	-96.3	25	-14.3	OH	93.1
H-2P-08	16.2	6.5	9.7	3.5	22	109	-92.8				
H-20P-09	16.7	10	6.7	15	23	107	-90.3	19	-2.3	OH	95.5
	16.7							74	-57.3	ML	110.5
	16.7							99	-82.3	OH	111.6
H-19P-09	12.9	7	5.9	8	13	120	-107.1				
H-18P-09	11.4	Not Measured		2	26	126	-114.6	25	-13.6	MH	93
	11.4							83	-71.6	ML	105.1
H-4-08	11.4	5	6.4	0	57	115	-103.6				
H-31P-09	11.7	3	8.7	14	21						
H-51P-09	12.1			16	18						
H-29P-09	13.0	6.5	6.5	14	14	115	-102.0				
H-08-09	12.6			10	10	118	-105.4				
H-09P-09	11.7	6.5	5.2	13.5	13.5	135	-123.4				
H-10P-09	31.6	25	6.6	23.5	28	137.5	-105.9				
H-11P-09	17.1	10.5	6.6	10	23	110	-92.9				
H-12P-09	16.4	8	8.4	9	45	120	-103.6				
Average	**15.1**	**8.3**	**7.3**	**11.4**	**19.3**	**117.1**	**-101.8**				
Range		**3 to 16.5**	**5.2 to 11.2**	**0 to 23.5**	**0 to 57**	**104 to 135**	**-89.8 to -123**				

Notes:
ft = feet
pcf = pounds per square foot

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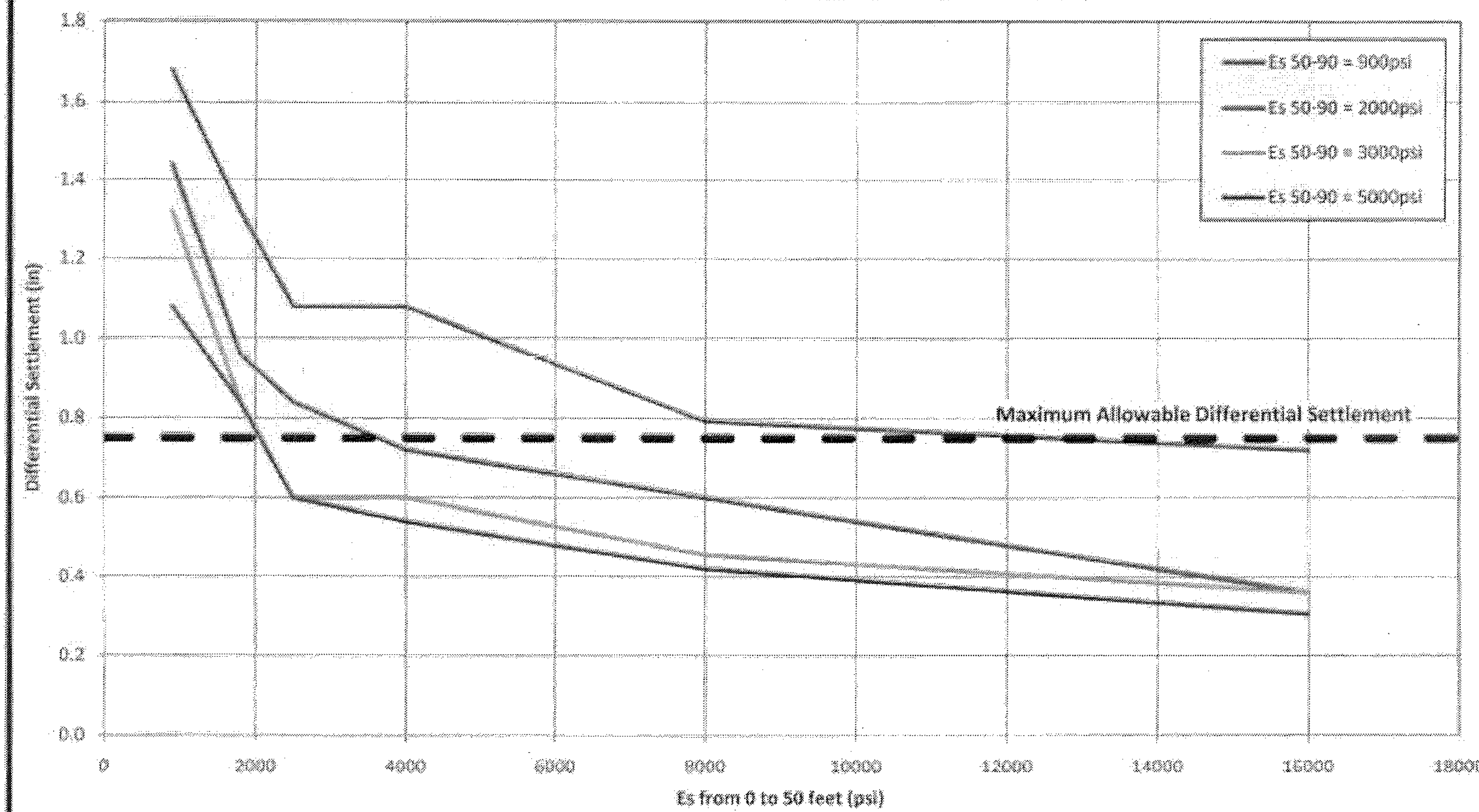
Pad Summary.xlsx 10/27/2009



NOTES

1. We performed the analyses using the computer program FLAC 3D (Itasca, 2008).
2. We analyzed a 3-ft-thick, uncracked concrete slab. The slab properties and critical moment were provided by HNTB on August 25, 2009.
3. The analysis assumes that the pontoon loads are applied instantaneously. Additional analyses will be needed to evaluate the construction sequence loading.

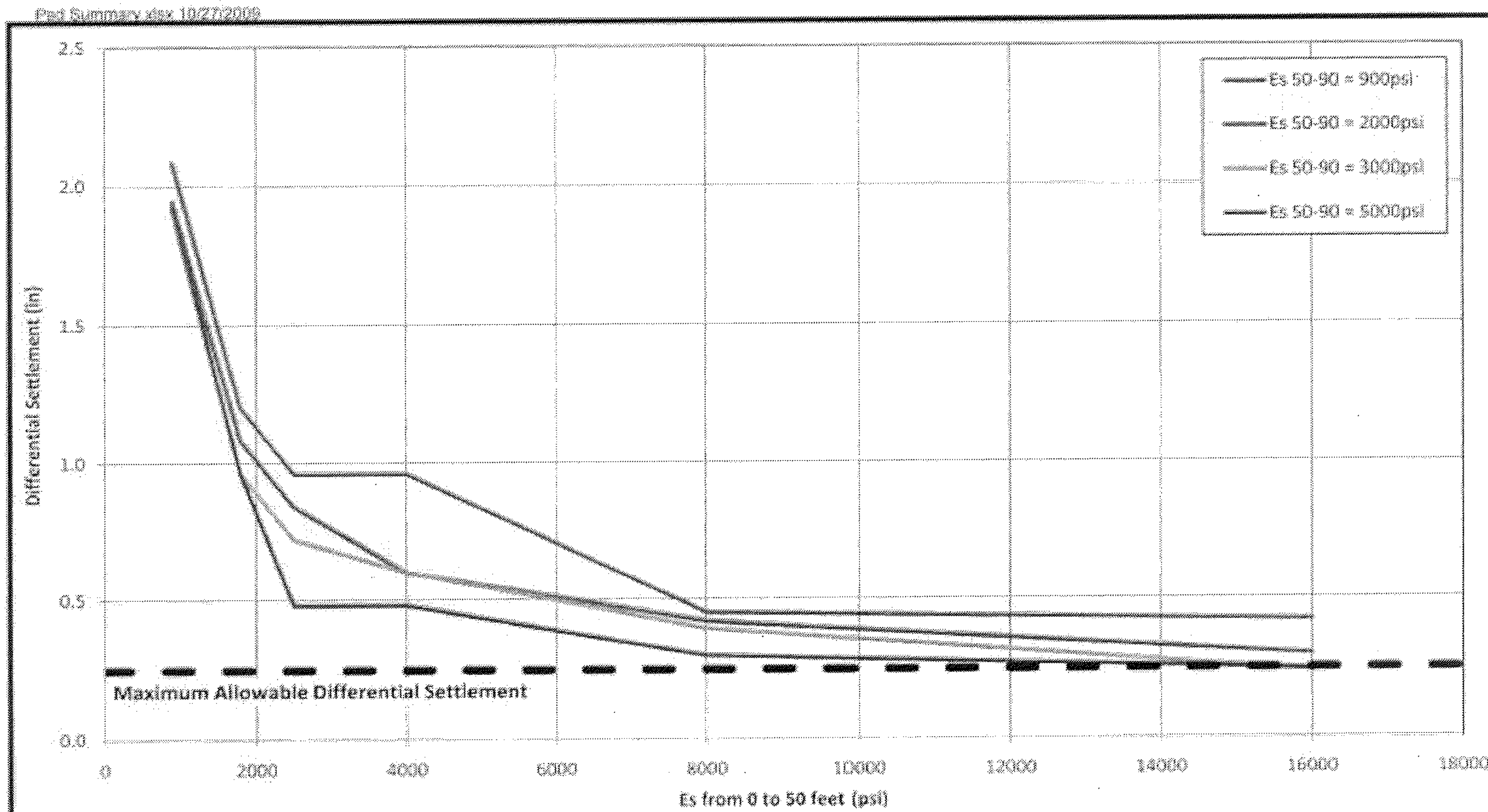
SR 520 Pontoon Casting Basin
Aberdeen, Washington

CENTER-LEFT DIFFERENTIAL SETTLEMENT
3-FT-THICK SLAB
TEST GROUND IMPROVEMENT

October 2009 21-1-21190-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

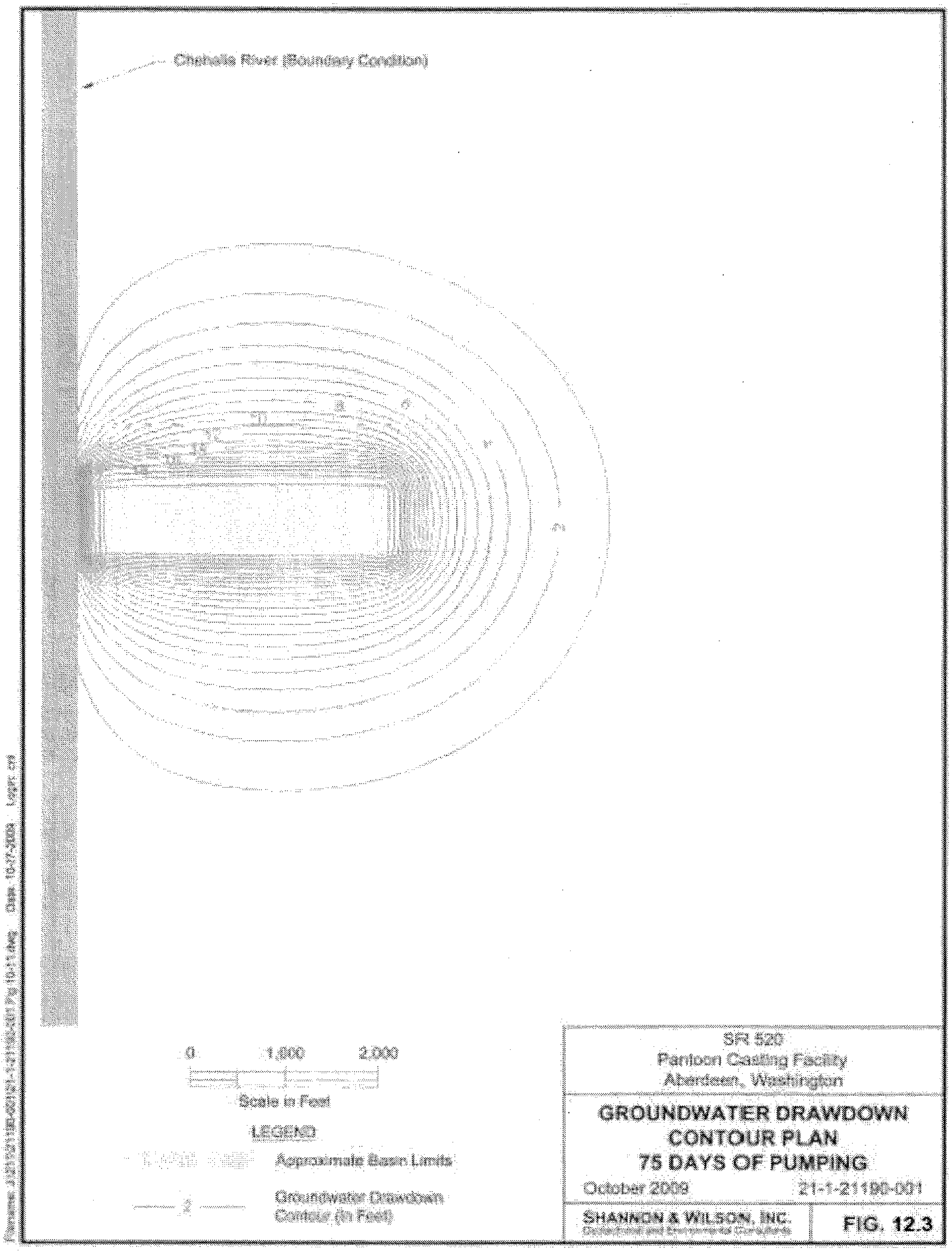
FIG 12.1

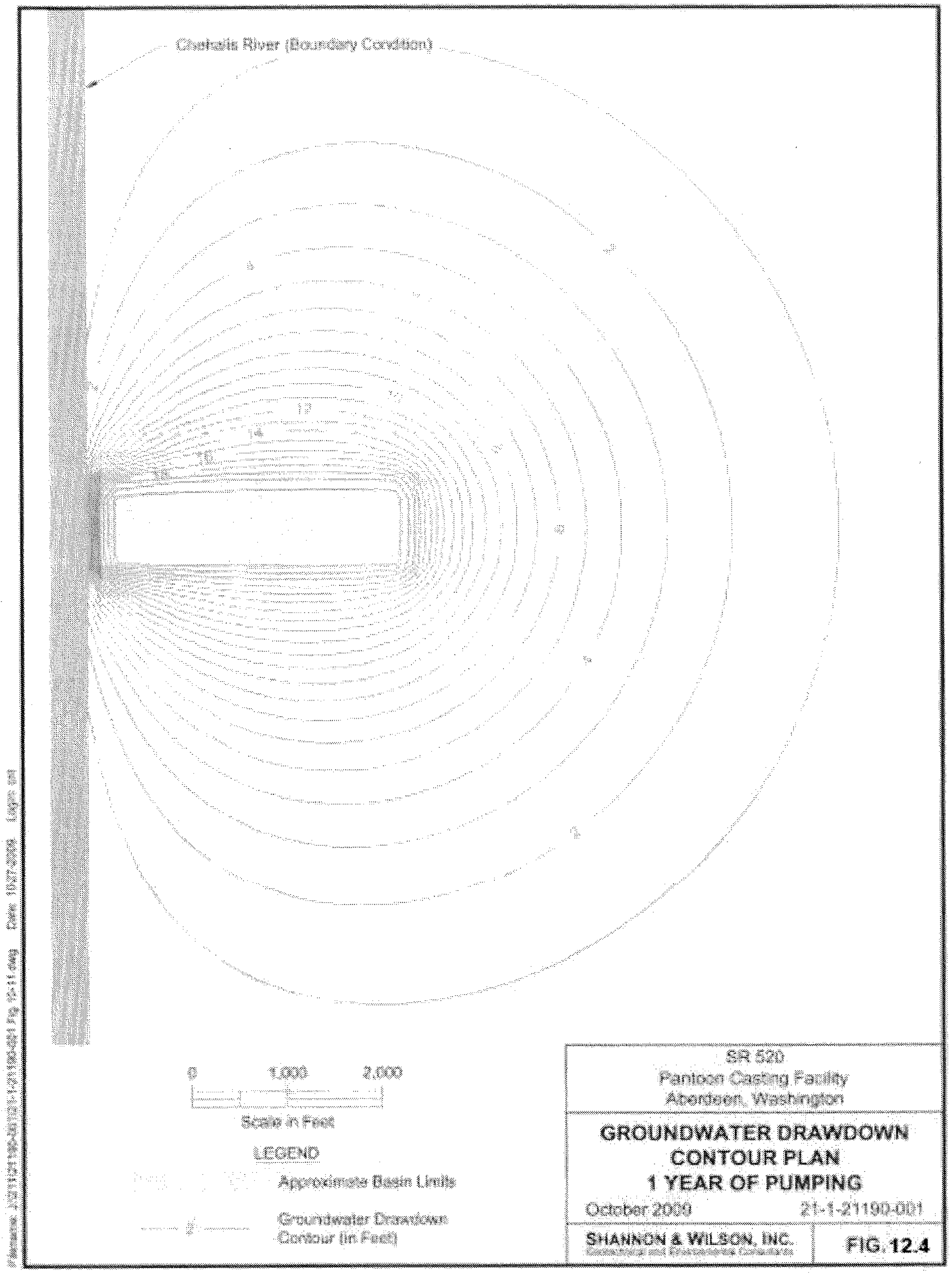


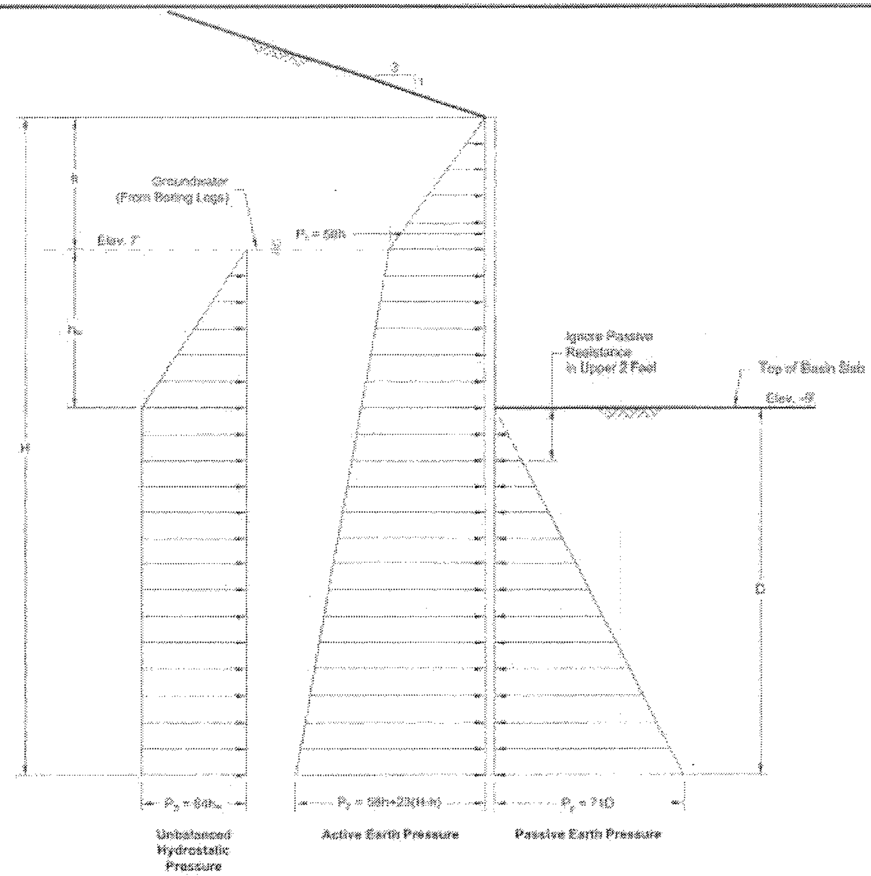
NOTES

1. We performed the analyses using the computer program FLAC 3D (Itasca, 2008).
2. We analyzed a 3-ft-thick, uncracked concrete slab. The slab properties and critical moment were provided by HNTB on August 25, 2009.
3. The analysis assumes that the pontoon loads are applied instantaneously. Additional analyses will be needed to evaluate the construction sequence loading.

SR 520 Pontoon Casting Basin Aberdeen, Washington	
CENTER-TOP DIFFERENTIAL SETTLEMENT 3-FT-THICK SLAB TEST GROUND IMPROVEMENT	
October 2009	21-1-21190-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 12.2


Chehalis River (Boundary Condition)
0
1,000
2,000
Scale in Feet
LEGEND
Approximate Basin Limits
Groundwater Drawdown Contour (in Feet)
SR 520
Pontoon Casting Facility
Aberdeen, Washington
GROUNDWATER DRAWDOWN CONTOUR PLAN
75 DAYS OF PUMPING
October 2009
21-1-21190-001
SHANNON & WILSON, INC.
FIG. 12.3


Chehalis River (Boundary Condition)
0
1,000
2,000
Scale in Feet
LEGEND
Approximate Basin Limits
Groundwater Drawdown Contour (in Feet)
SR 520
Pontoon Casting Facility
Aberdeen, Washington
GROUNDWATER DRAWDOWN CONTOUR PLAN
1 YEAR OF PUMPING
October 2009
21-1-21190-001
SHANNON & WILSON, INC.
FIG. 12.4



NOTE
Passive earth pressures include a factor-of-safety=1.5

Not to Scale

SR 520
Pontoon Casting Facility
Aberdeen, Washington

RECOMMENDED PRESSURES FOR TOE WALL IN BASIN

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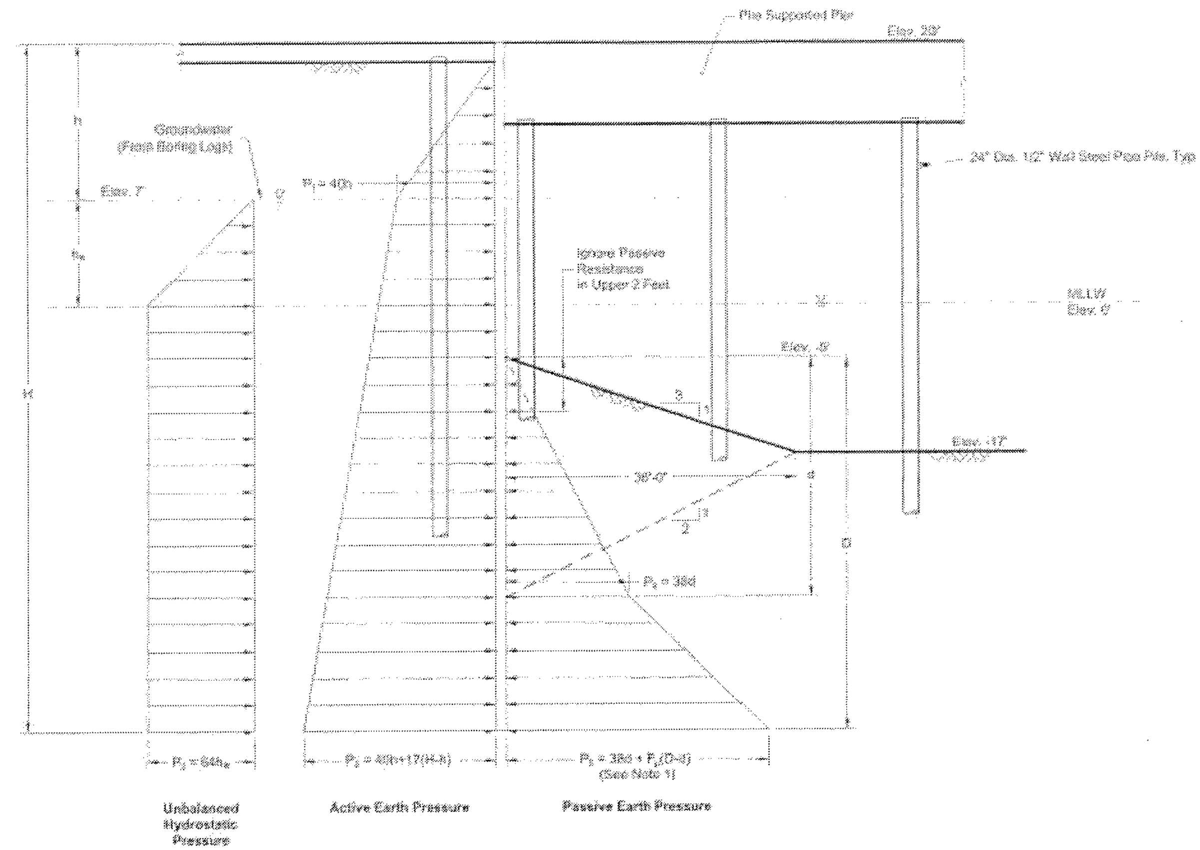
FIG 12.5



NOTES

1. If point of zero moment is below d, use $F_p = 71$; if the point is above d, use $F_p = 38$.
2. All pressures in psf.
3. Passive earth pressures include a factor of safety = 1.5

Not to Scale

SR 520
Pontoon Casting Facility
Aberdeen, Washington

RECOMMENDED PRESSURES FOR BULKHEAD

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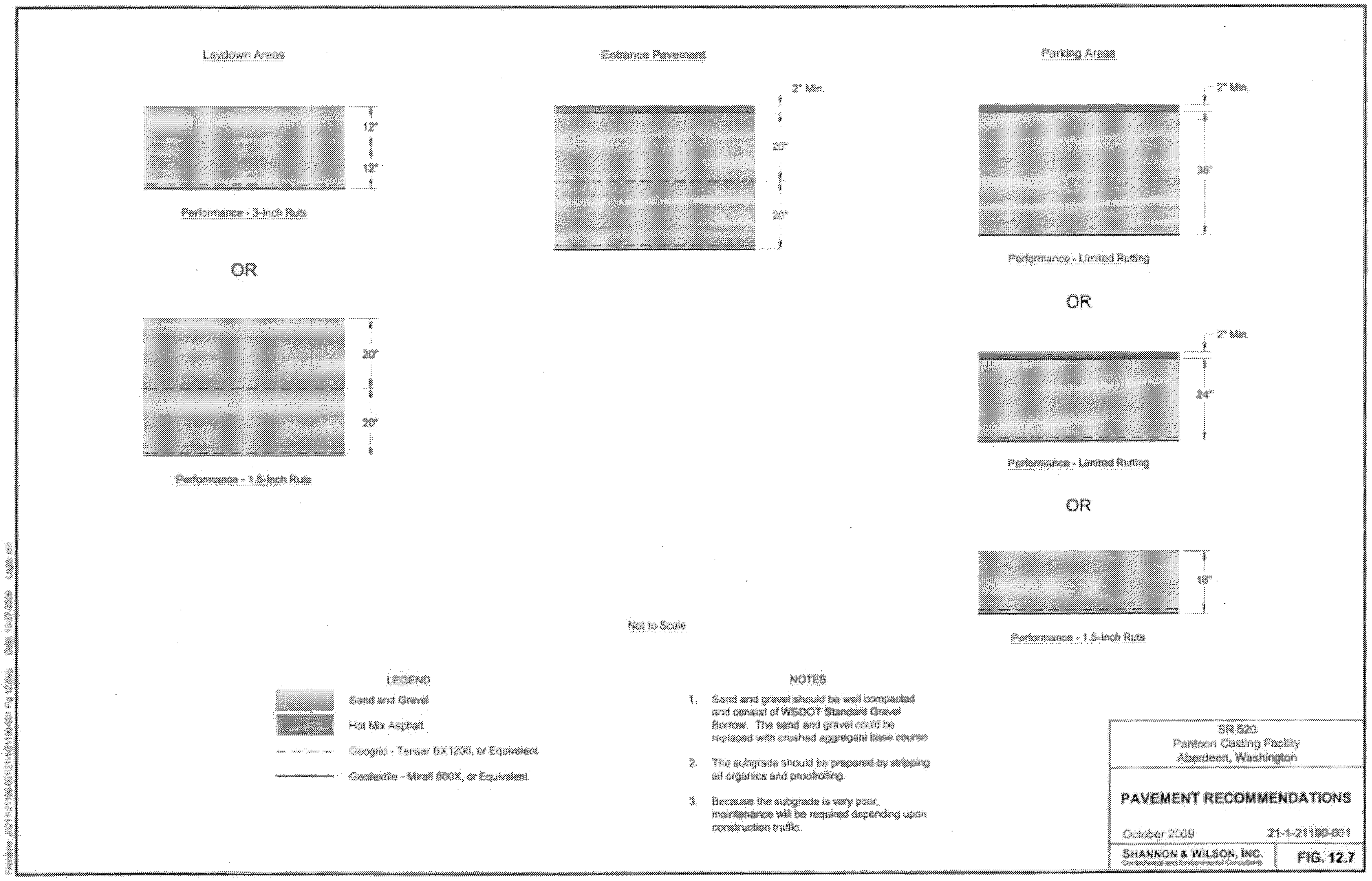
FIG 12.6


Laydown Areas
12"
12"
Performance - 3-inch Ruts
OR
20"
20"
Performance - 1.5-inch Ruts
Entrance Pavement
2" Min.
20"
20"
Parking Areas
2" Min.
36"
Performance - Limited Rutting
OR
2" Min.
24"
Performance - Limited Rutting
OR
18"
Performance - 1.5-inch Ruts
Not to Scale
LEGEND
Sand and Gravel
Hot Mix Asphalt
Geogrid - Tensar BX1200, or Equivalent
Geotextile - Mirafi 600X, or Equivalent
NOTES
1. Sand and gravel should be well compacted and consist of WSDOT Standard Gravel Borrow. The sand and gravel could be replaced with crushed aggregate base course
2. The subgrade should be prepared by stripping all organics and proofrolling.
3. Because the subgrade is very poor, maintenance will be required depending upon construction traffic.
SR 520
Pontoon Casting Facility
Aberdeen, Washington
PAVEMENT RECOMMENDATIONS
October 2009
21-1-21190-001
SHANNON & WILSON, INC.
FIG. 12.7

Appendix A

Basin Floor & Walls As-Built Drawings
(Reserved)

Appendix B

Power Crowder As-Built Drawings
(Reserved)